LIVE-LOAD STRESSES

IN

RAILWAY BRIDGES

WITH

FORMULAS AND TABLES

BY

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PREFACE

Stresses caused by moving concentrated loads are treated in this book by the combined use of influence lines and algebraic methods. The influence line is connected by this treatment with tables of moment sums and load sums in a new and entirely practical manner.

The heart of the text is contained in equations (7) and (8). These give an easy and exact solution of the maximum live-load stresses in any structure whose influence lines can be drawn, replacing, for the more complicated structures, such as cantilever and swing bridges, arches, etc., the old method of placing the wheel loading by trial and scaling the influence-line ordinates under the loads.

A second feature of the text is the application of equations (7) and (8) to the simpler structures, such as girder bridges (with and without panels), pier reactions, and Pratt trusses (with inclined and horizontal chords), in which these equations are transformed and simplified to meet the requirements of these ordinary cases. This leads to a series of simple formulas to meet the needs of every-day designing. To illustrate the application of these formulas, fully worked-out examples are given.

The text is supplemented by a very complete set of tables, the usefulness of which is at once apparent. The greater part of the matter in these tables is new. A table similar to Table 3 was made by Mr. Josiah Gibson, C.E., and published in the *Engineering News*, June 21, 1906; and a table similar to Table 11 is given by Mr. J. P. J. Williams in the *Engineering News* of Oct. 1, 1914. Tables similar to Tables 6, 8, and 9 are found in the "Structural Engineers' Handbook" by Dean Milo S. Ketchum and in the "Design of Steel Bridges" by Mr. F. C. Kunz.

The author wishes to acknowledge his indebtedness to the American Bridge Company for material assistance, and in particular to Mr. O. E. Hovey, Assistant Chief Engineer of this company, for his encouragement and help. The author also desires to acknowledge the valuable suggestions made in the revision of the original text by Professor F. H. Constant, of the Civil Engineering Department of Princeton. To Professor William H. Burr of Columbia University, the writer is permanently indebted for the logical and thorough instruction received from him as a student.

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LIVE-LOAD STRESSES

ARTICLE I.

INFLUENCE LINES. DEFINITION AND USES.

INFLUENCE lines are useful in determining the position of live load on a bridge to produce maximum effect. They offer also a convenient method of deriving general algebraic formulas for stresses and rules for maximum when the general relations between influence lines and algebraic formulas are once understood; and in the case of the more complex problems of skew bridges, arches, cantilever bridges, etc., the influence lines themselves serve as a most direct method for the determination of the maximum live-load stresses.

An influence line may be defined as a line showing the variation in any function caused by a single *unit* load as it moves across the bridge. Vertical loads only will be considered. The function may be a reaction, bending moment, shear, stress, deflection, or any quantity whatsoever at a given part of a bridge, provided that its value is a function of the position of the unit load on the bridge.

Refer to Fig. 1a. Consider the span AB, and let Z be any function at the fixed position C on the span L. If the load unity moves across the span AB and the value of Z be calculated for each position of the unit load and its value z plotted below the corresponding position of this load as an ordinate from a horizontal base line, the locus of the plotted points will be the influence line for Z. For example, if Z be the bending moment at the fixed section C in a beam of span L, the influence line will be as shown in Fig. 1b. In plotting influence lines, ordinates repre-

senting positive quantities are plotted above the base line; and negative, below. In case the influence line consists of several straight segments, it is necessary to determine the value of the ordinates only where the influence line has a change of direction; *i.e.*, at the *salient points*. For example,

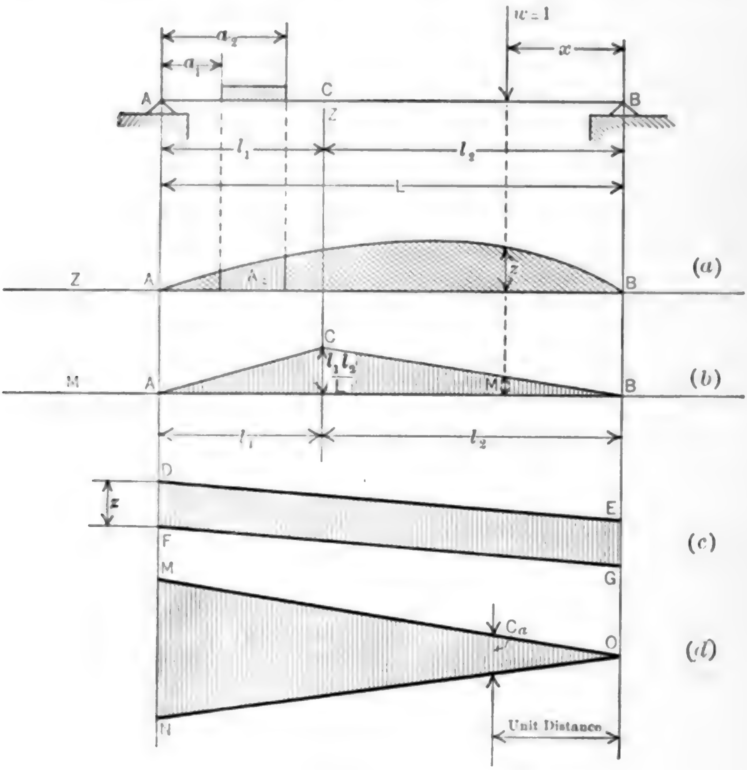

FIG. 1.

the points A, C, and B are the salient points of the influence line in Fig. 1b.

The value of Z caused by a single load w is equal to wz, if z is the influence ordinate below w. The value of Z caused by a series of loads w_1, w_2, w_3, etc., is

$$Z = w_1z_1 + w_2z_2 + w_3z_3 + \dots = \Sigma wz \quad . \quad . \quad . \quad (1)$$

where z_1, z_2, z_3, etc., are the influence ordinates below the corresponding loads. It will be convenient to speak of such a quantity as wz as an *ordinate-load product.*

Formula (1) therefore may be expressed thus:

Z = *Sum of ordinate-load products.*

The area between the influence line and the base line is called the *influence area.* It may be shown that the value of Z caused by a uniform load on the bridge is proportional to the area A_s of the influence line between the ordinates at the extremities of the uniform load. If the uniform load in Fig. 1a has an intensity of q per unit of length, the load in the length dx equals $q\,dx$, and the influence of this elementary load on the value of Z is $zq\,dx$, where z is the influence ordinate below $q\,dx$. Summing up for the length of the uniform load,

$$Z = q\sum_{a_1}^{a_2} z dx = qA_s \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

If a series of equal loads w is on the span, the value of Z is

$$Z = \Sigma wz = w\Sigma z \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

If a series of unequal loads, w_1, w_2, etc., is multiplied by the corresponding ordinates of an influence line or a portion of an influence line which has a constant ordinate z, as in Fig. 1c, the value of Z is

$$Z = z(w_1 + w_2 + \ldots) = z\Sigma w = zW \quad . \quad . \quad . \quad (4)$$

where W equals the sum of these loads.

If a series of unequal loads is multiplied by the corresponding ordinates of an influence line or a portion of an influence line consisting of two diverging lines, as shown in Fig. 1d, the value of Z, or the sum of the ordinate load products, and the rate at which Z varies as the loading advances, are given by the two theorems that follow. The *slope* of a line is defined at the beginning of Art. 2.

Theorem I.

The sum of the ordinate-load products between two diverging lines equals the difference between the *slopes* of the two lines multiplied by the sum of the moments of the loads about the intersection of these lines.

In symbols, this is stated as

$$Z = C_a M_a \quad \ldots\ldots\ldots \quad (5)$$

Theorem II.

The rate at which the sum of the ordinate-load products between the two diverging lines increases as the loading moves away from the intersection of these lines equals the difference between the *slopes* of the two lines multiplied by the sum of the loads.

In symbols, this is stated as

$$\frac{dZ}{dx} = C_a W_a = \frac{d(C_a M_a)}{dx} = C_a \frac{dM_a}{dx} \quad \ldots\ldots \quad (5a)$$

The proofs of these theorems follow in the next article.

ARTICLE II.

SUM AND RATE OF VARIATION OF ORDINATE-LOAD PRODUCTS BETWEEN THE TWO DIVERGING LINES.

Consider the diverging lines DAB and AC in Fig. 2. Use the following notation:

w = any vertical load.

z = ordinate below w in the angle BAC.

$Z = \Sigma w_n z_n$ = sum of ordinate-load products.

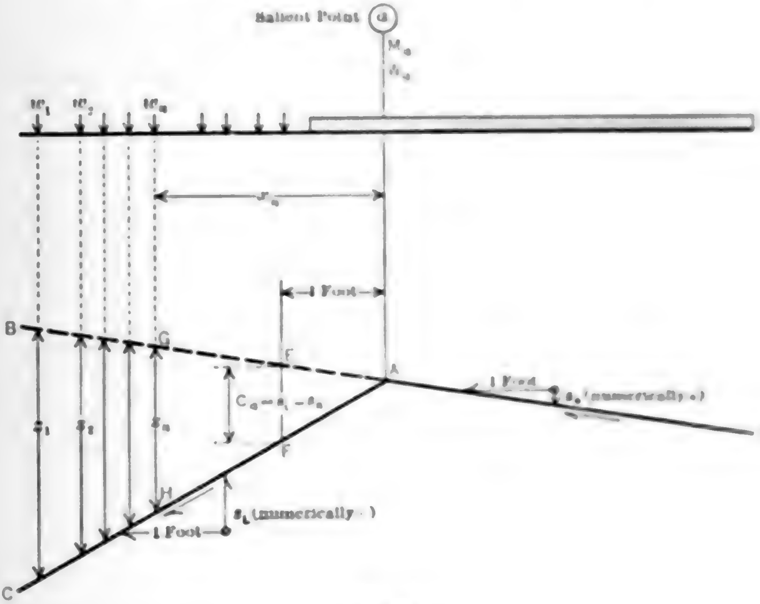

Fig. 2.

$M_a = \Sigma w_n x_n$ = moment sum of all loads to left of Aa about A.

$W_a = \Sigma w_n$ = load sum of all loads to left of Aa.

s_R = slope of line DA = tangent of angle which DA makes with the horizontal.

s_L = slope of line AC = tangent of angle which AC makes with the horizontal.

$C_a = \frac{z_n}{x_n} = (s_L - s_R)$ = length of ordinate unit distance from A.

Slopes are counted numerically positive when upward to the left. The sign of C_a (called the coefficient at salient point A) is, accordingly, negative when AC diverges below DA produced to the left of A. The value of C_a may be determined graphically as $\frac{z_n}{x_n}$ or it may be figured algebraically as $(s_L - s_R)$.

Proof of Theorem I, or that $Z = C_a M_a$.

Consider the load w_n distant x_n from the salient point a. By the similar triangles AEF and AGH,

$$\frac{C_a}{1.00} = \frac{z_n}{x_n}, \text{ or } z_n = C_a x_n.$$

Therefore,

$$w_n z_n = C_a w_n x_n. \quad \ldots \ldots \ldots \quad (A)$$

Summing up all of the ordinate-load products,

$$Z = \Sigma w_n z_n = C_a \Sigma w_n x_n = C_a M_a. \quad \ldots \ldots \quad (5)$$

Proof of Theorem II, or that $\frac{dZ}{dx} = C_a W_a$.

From equation (A) above, the increase in the ordinate-load product $w_n z_n$ for an advance dx_n of the load is

$$w_n dz_n = C_a \cdot w_n \cdot dx_n.$$

Summing up the increases of all the ordinate-load products and noting that dx is the same for all loads,

$$dZ = \Sigma w_n dz_n = C_a dx \cdot \Sigma w_n = C_a \cdot W_a \cdot dx.$$

Dividing by dx, $\frac{dZ}{dx} = C_a W_a = \frac{d(C_a M_a)}{dx} = \frac{C_a dM_a}{dx} \quad \ldots \quad (5a)$

ARTICLE III.

SUM AND RATE OF VARIATION OF ORDINATE-LOAD PRODUCTS FOR ANY INFLUENCE LINE. POSITION OF LOADING FOR MAXIMUM LIVE-LOAD STRESS.

An influence line of a general type is shown in Fig. 3, this one in particular being for the member U'_3L_4 of the

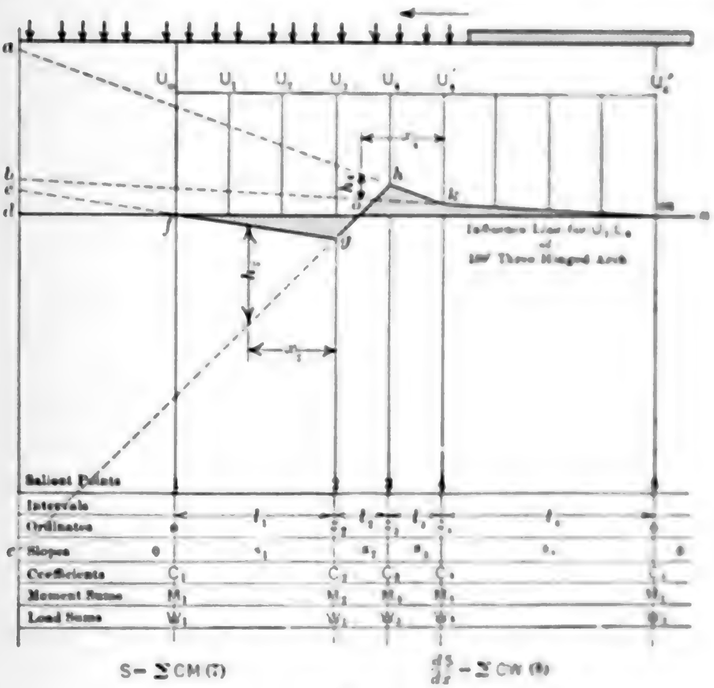

Fig. 3.

arch shown in Fig. 15. It is assumed that the ordinates at all salient points and the intervals between these points are known. Ordinates and slopes are counted positive or negative as already defined. The *slope of any segment* of the

influence line equals the ordinate at the left minus the ordinate at the right end of this segment divided by the corresponding interval. The *coefficient C at any salient point* equals the slope of the segment at the left minus the slope of the segment at the right of this point. The subtractions in each case are made algebraically.

It should be remembered, as has already been pointed out in Art. 2, that the value of any coefficient C may also be measured graphically from an influence line which has been drawn to scale. For example, in Fig. 3 the value of the coefficient $C_2 = \frac{h_2}{x_2}$ and $C_4 = \frac{h_4}{x_4}$.

The algebraic calculation of the coefficients at all salient points of the influence line in Fig. 3 is given below. If it be assumed that this influence line has been drawn to scale, the signs of the numerical values of the slopes and coefficients will be as given in the parentheses.

$$s_1 = \frac{0 - z_2}{l_1} \quad (+) \qquad C_1 = 0 - s_1 \quad (-)$$

$$s_2 = \frac{z_2 - z_3}{l_2} \quad (-) \qquad C_2 = s_1 - s_2 \quad (+)$$

$$s_3 = \frac{z_3 - z_4}{l_3} \quad (+) \qquad C_3 = s_2 - s_3 \quad (-)$$

$$s_4 = \frac{z_4 - 0}{l_4} \quad (+) \qquad C_4 = s_3 - s_4 \quad (+)$$

$$C_5 = s_4 - 0 \quad (+)$$

A numerical evaluation of the slopes and coefficients for this influence line is given in Fig. 15 of Art. 8, which the reader should check in order to understand completely the method of procedure. These coefficients should also be checked by the graphical method as already explained. For example, in Fig. 15 the value of $C_2 = \frac{2.59}{30} = .0863$.

It will be noted in the algebraic calculation of the coefficients C at all salient points that each slope enters once

as positive and once as negative. Therefore the sum of all coefficients equals zero.

$$\Sigma C = 0. \qquad (6)$$

This formula serves as a check on the values of the coefficients which have been determined either by calculation or by graphical measurement.

The general formulas for the sum of the ordinate-load products for any influence line (viz., with several salient points such as the one shown in Fig. 3) may be arrived at by considering the two contiguous sloping sides of the influence line meeting at each salient point as two diverging lines. The entire influence line is thus made up of pairs of diverging lines (see Fig. 3) to each pair of which formula (5) may be directly applied. Thus in Fig. 3,

Ordinate-load	products	in	dfc	$=$	C_1M_1	$(-)$
"	"	"	cge	$=$	C_2M_2	$(+)$
"	"	"	eha	$=$	C_3M_3	$(-)$
"	"	"	akb	$=$	C_4M_4	$(+)$
"	"	"	bmd	$=$	C_5M_5	$(+)$

The signs of the CM's are + or − according to the signs of the coefficients, for the M's are always positive. Summing up the above equations and observing that the ordinate-load products cancel one another except between the influence line $fghkm$ and its base line fom, it follows that the sum of the ordinate-load products for the influence line, or the live-load stress, is

$$S = C_1M_1 + C_2M_2 + \ldots = \Sigma CM. \qquad (7)$$

The letter S represents in general any stress or sum of ordinate-load products for any influence line, while Z stands for the sum of ordinate-load products for any geometrical figure.

The rate at which S varies as the load advances a distance dx equals

$$\frac{dS}{dx} = \frac{d(C_1M_1)}{dx} + \frac{d(C_2M_2)}{dx} + \text{Etc.}$$

But by formula (5a) this becomes

$$\frac{dS}{dx} = C_1W_1 + C_2W_2 + \ldots = \Sigma CW. \quad . \quad . \quad . \quad (8)$$

W_1, W_2, etc., = sum of all of the loads to the left of points 1, 2, etc., respectively, whether on the span or not.

M_1, M_2, etc., = moment of the same loads about points 1, 2, etc., respectively, whether on the span or not.

The above formulas (6), (7), and (8) apply equally well when the loading is headed from left to right instead of from right to left, the latter being the more usual way. In applying these formulas, however, it will save confusion not to reverse the loading, but to turn the influence line end for end, for this operation changes neither the values nor the signs of the coefficients C.

The stress $S = \Sigma CM$ is related to its derivative $\frac{dS}{dx} = \Sigma CW$ in the same way that any function is related to its derivative. Thus, if the value of $\frac{dS}{dx}$ passes through zero as the loading advances, the stress itself may have reached any one of four conditions; namely,

1. Numerically maximum positive value.
2. " minimum " "
3. " maximum negative "
4. " minimum " "

In practice it is desirable to find the positions of loading to satisfy the first and third conditions. This may be done by proceeding as directed below. It is assumed in stating the following rules that the live load is advancing from right to left. In case the live load advances from left to right, the wheel will be tried first to the left and

then to the right of a salient point. In other words, dx is always an increment in the same direction as the loading advances.

Rule 1.—To determine the position of loading to give a maximum positive stress, place the live load on the part of the bridge corresponding to the positive portion of the influence line. Try a wheel first immediately to the right of a salient point that has a *negative* coefficient and then just to the left of this point. Calculate the value of $\frac{dS}{dx} = \Sigma CW$ for each of these successive positions of loading. If the sign of $\frac{dS}{dx}$ changes from + to −, a position of loading for maximum positive stress is determined.

Rule 2.—To determine the position of loading to give a numerically maximum negative stress, place the live load on that part of the bridge corresponding to the negative portion of the influence line. Try a wheel first immediately to the right of a salient point that has a *positive* coefficient and then just to the left of this point. Calculate the value of $\frac{dS}{dx} = \Sigma CW$ for each of these successive positions of loading. If the sign of $\frac{dS}{dx}$ changes from − to +, a position of loading for numerically maximum negative stress is determined.

It will be noted that the negative coefficients C occur at those salient points where the angles of the influence line point *upward*, while the positive coefficients C occur at those salient points where the angles point *downward*.

It is unnecessary to seek a position of loading for *maximum positive* stress by placing a wheel successively to the right and to the left of any salient point which has a *positive* coefficient; for if $\frac{dS}{dx} = \Sigma CW$ be + when the wheel is to the right of this point, it would have a still larger +

value when the wheel is to the left of the point. A change, therefore, of $\frac{dS}{dx}$ from + to − would not result. Similarly, it may be shown to be unnecessary to seek a numerically *maximum negative* stress by trying wheels at any salient point which has a *negative* coefficient.

Formulas (7) and (8) are the general formulas for the solution of the sum of the ordinate-load products of an influence line and the rate of change of this sum, and are applicable to any form of influence line. They give at once a definite solution of the position of a set of loads producing maximum positive and negative stresses in any member of any truss or girder for which an influence line can be drawn and the values of such stresses. The method is particularly advantageous in the case of statically indeterminate structures, such as two-hinged and no-hinged arches, swing bridges, continuous girders, etc., where general analytical criteria for the positions of loads producing maximum stresses cannot readily be expressed and where such maximum stresses have had to be found by assuming positions of loadings and scaling the influence-line ordinates under all the loads, a laborious process and one open to much liability of mechanical inaccuracy.

In applying the present method to the simple forms of girders and trusses (viz., the statically determinate structures where the ordinates of the influence lines are readily expressible algebraically) it will generally be more convenient to transform formulas (7) and (8) in each case whereby the coefficients C may be expressed in terms of the geometric proportions of the truss or girder. This, in the following articles (4 to 7 inclusive), we shall proceed to do for the case of girder bridges (with and without panels), pier reactions, and through Pratt trusses with curved or horizontal chords. The general method will, however, be applied *directly* to the case of the three-hinged arch in Art. 8, which will serve as a typical example of the application of the method to any influence line.

ARTICLE IV.

GIRDER BRIDGE WITHOUT PANELS.

In Fig. 4 is shown a girder bridge without panels. The live load has advanced beyond the span, this being the

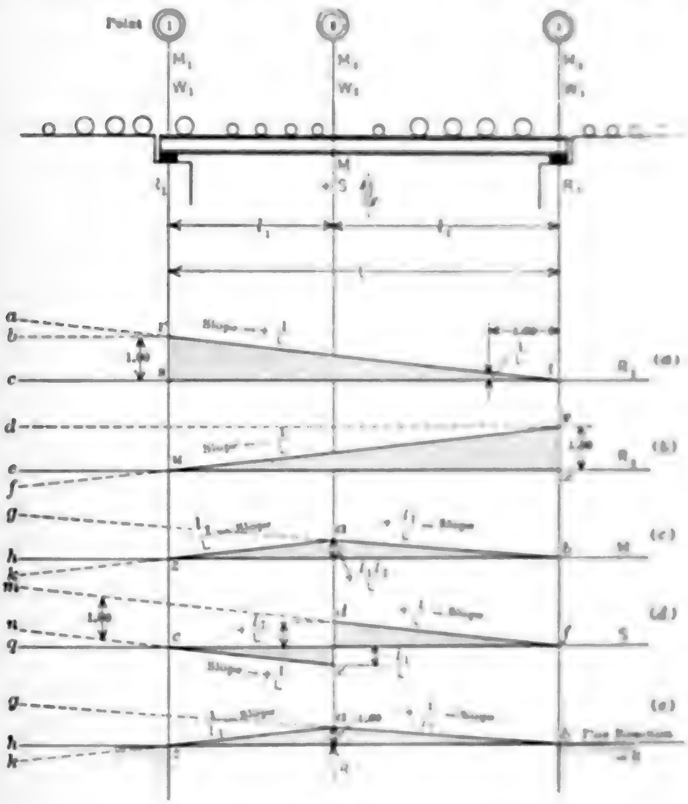

Fig. 4.

most general case. Formulas for the end reactions and for the bending moment and shear at any section will be developed.

The influence line for R_1 is shown in Fig. 4a. The sum of the ordinate-load products within the shaded area *rst* equals the end reaction R_1, which at the same time is the *end shear* at R_1.

From Fig. 4a,

	Ordinate-load	products	in	*rst*	=
	"	"	"	"	*atc*
—	"	"	"	"	*arb*
—	"	"	"	"	*brsc.*

By using formulas (4) and (5), this equation becomes

$$R_1 = \frac{1}{L} M_3 - \frac{1}{L} M_1 - W_1 = \frac{M_3 - M_1}{L} - W_1 \quad . \quad . \quad (9)$$

Any value of M or W may be read directly from Table 2 for the standard loadings given in Table 1. For example, in Fig. 4, if $l_1 = 10'$, $l_2 = 30'$, and w_1 of Cooper's $E50$ has advanced $14'$ beyond the left end of the span, we have from Table 2,

At 1, $14'$ from w_1, $M_1 = 350.0^{K'}$	$W_1 = 62.50^K$
At 2, $24'$ from w_1, $M_2 = 1150.0$	$W_2 = 112.50$
At 3, $54'$ from w_1, $M_3 = 5435.0$	$W_3 = 177.50$

The formula for R_2 is developed as for R_1, the method of writing the second member of the first equation being abbreviated in a way readily understood. From the influence line in Fig. 4b, and the formulas (4) and (5),

R_2 = Ordinate-load products in (*dvxe* − *dvf* + *fue*)

Or

$$R_2 = W_3 - \frac{1}{L} M_3 + \frac{1}{L} M_1 = W_3 - \frac{M_3 - M_1}{L} \quad . \quad (9a)$$

The sum of the reactions R_1 and R_2 as given by (9) and (9a) equals $W_3 - W_1$, or the sum of the loads on the bridge.

From the influence line in Fig. 4c and formulas (5) or (7), the equation for bending moment may be written:

M = Ordinate-load products in (*gbh* − *gak* + *kzh*).

Or

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \qquad (10)$$

Formula (10) readily follows, likewise, from the general formula (7), $S = C_1M_1 + C_2M_2 + C_3M_3 = \Sigma CM$.

For example, in the case of the bending moment at point 2 in Fig. 4,

$$C_1 = 0 + \frac{l_2}{L}$$

$$C_2 = -\frac{l_2}{L} - \frac{l_1}{L} = -1$$

$$C_3 = \frac{l_1}{L} - 0$$

Whence $$M = \frac{l_2}{L} M_1 - M_2 + \frac{l_1}{L} M_3 \qquad (10a)$$

Taking the derivative of M with respect to the advance dx of the loading toward the left or using formula (8) directly, the rate of variation of the bending moment is

$$\frac{dM}{dx} = \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \qquad (11)$$

All positions for maximum M may be found by trying wheels at point 2 as directed by Rule 1 of Art. 3. In applying this rule the simultaneous shifting of other wheels of the rigid loading from right to left of points 1 and 3 as a wheel is shifted from right to left of point 2, must be taken into account by substituting in formula (11) the corresponding changed values of W_1 and W_3. It is to be remembered, as stated in Art 3, that it is entirely unnecessary to try wheels at points 1 and 3.

From the influence line in Fig. 4d, the formula for the intermediate shear S follows by applying formulas (4) and (5):

S = Ordinate-load products in

$$(\lfloor mfq - mden - \lfloor ncq)$$

Or

$$S = \frac{1}{L} M_3 - W_2 - \frac{1}{L} M_1 = \frac{M_3 - M_1}{L} - W_2 \quad . \quad (12)$$

There is one more thing to be borne in mind in calculating maximum bending moments in a girder bridge without panels: it is the rule for finding the section where the *absolute maximum bending moment* occurs. The rule is often spoken of as the "*centre of gravity rule,*" and may be stated as follows:

The bending moment under any given wheel becomes maximum when the centre of the span bisects the distance from the wheel in question to the centre of gravity of the loading on the span.

In the practical application of this rule, the procedure is first to find the wheel which gives maximum bending moment at the centre of the span and then to shift this wheel so that the bending moment beneath it becomes an absolute maximum according to the centre of gravity rule. For the usual standard loadings the maximum centre moment closely approximates the absolute maximum bending moment for the spans greater than 70 feet.

The proof of the centre of gravity rule follows. Refer to Fig. 5. Assume that it has been found by trial that the wheel w_n gives the maximum centre moment. The general case where load has advanced beyond the span is taken. In order to get an *absolute maximum* bending moment under w_n, this wheel must be shifted a certain distance from the centre. Let such position be distance y from R_1. The sum of the loads on the span is called P_2 and equals $(W_3 - W_1)$. The centre of gravity of the loads P_2 is distance $\bar{x}$ from R_2. The sum of the loads on the span to the left of w_n is called P_1, and their centre of gravity is at the fixed distance b from w_n.

Taking moments about R_2,

$$R_1 = \frac{P_2 \bar{x}}{L}$$

Therefore,

$$M = R_1 y - P_1 b = \frac{P_2 \bar{x}}{L} y - P_1 b.$$

In this equation for M, the only variables are $\bar{x}$ and y. Therefore, M will be a maximum when the product $\bar{x}y$ is maximum. Note, however, that the sum

$$\bar{x} + y = (L - a) = \text{constant}.$$

If two variables have a constant sum, their product is maximum when the two variables are equal. Therefore, M is maximum when $\bar{x} = y$. But when $\bar{x} = y$, the distance from w_n to the centre of gravity of the loading is bisected

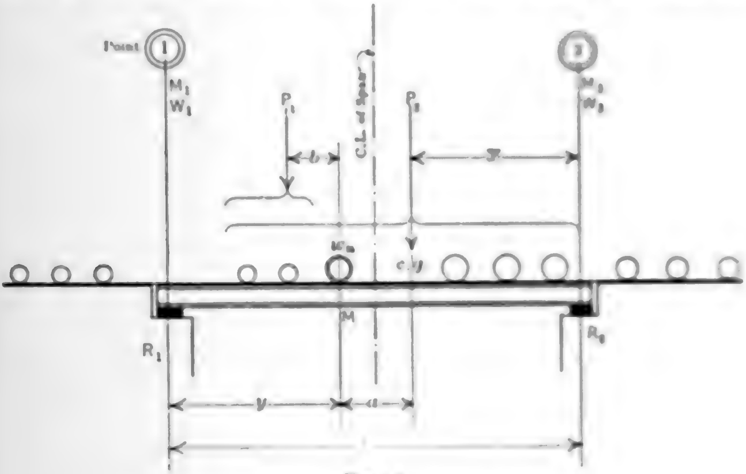

FIG. 5.

by the centre of the span. This proves the centre of gravity rule.

In order to apply this rule, a general expression for $\bar{x}$ is needed.

Since $R_1 = \dfrac{P_2 \bar{x}}{L}$ it follows that $\bar{x} = \dfrac{R_1 L}{P_2}$. Substitute the value of R_1 from formula (9), and the value $(W_2 - W_1)$ for P_2.

$$\bar{x} = \frac{M_2 - M_1 - LW_1}{W_2 - W_1} \quad \text{. . . . . (13)}$$

In the special case where the loading has not advanced beyond the left end of the span, M_1 and W_1 equal zero and x becomes

$$\bar{x} = \frac{M_3}{W_3} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (13a)$$

Problems relating to a girder bridge without panels will now be given to illustrate the application of the above formulas and the use of some of the tables following the text.

Problem.—Given a 40-foot deck-girder bridge consisting of one girder per rail. Use Cooper's $E50$ loading. Find the maximum shear at the end, quarter point, and centre. Determine also the maximum bending moment at the quarter point and at the centre, and the absolute maximum bending moment. All values are to be given *per rail.*

Solution.—Table 5 following the text gives the position of Cooper's loadings for maximum end shear. This table is the result of the solution of end shears for a large number of spans. As a general rule, however, it is safe to assume that w_2 of Cooper's and similar loadings will always give the maximum end or intermediate shear when placed immediately to the right of the given section, the live load being headed toward the left. The exceptions in Table 5 to this general rule are not of prime importance, for the actual value of the shear when w_2 is used is sufficiently close to the maximum even in the exceptional cases. There is no satisfactory criterion for determining the position of loading for maximum shear in girder bridges without panels, for it is as easy to calculate the actual values of the shears for the successive positions of loading as it is to apply any criterion. In the case of bending moment, however, time is saved by using the criterion.

Maximum End Shear.

Use formula (9), $R_1 = \frac{M_3 - M_1}{L} - W_1$. Place wheel 2

of Cooper's *E*50 immediately to right of R_1. Take the values of moment and load sums for Cooper's *E*50 from Table 2.

$$\text{Maximum end shear} = \frac{4370 - 100}{40} - 12.5 = 94.3^k.$$

Maximum Shear at Quarter Point.

Use formula (12) with w_1 at quarter point.

$$S = \frac{M_1 - M_1}{L} - W_1$$

$$S \text{ at } \tfrac{1}{4} \text{ point} = \frac{2838.75 - 0}{40} - 12.5 = 58.5^k.$$

Maximum Shear at Centre.

Using formula (12) with w_1 at centre.

$$S \text{ at centre} = \frac{1600 - 0}{40} - 12.5 = 27.5^k.$$

The values for the shears are given in Kips, or thousand of pounds. A comparison of the above shears with those in Table 7 shows agreement of results.

Maximum Bending Moment at the One-Quarter Point.

First compute successive pairs of values for $\frac{dM}{dx}$ for different wheels, first placed to the right and then to the left of the quarter point. A change of sign from + to − indicates a wheel that gives a maximum. Use formula (11).

$$\frac{dM}{dx} = \frac{l_1}{L} W_2 + \frac{l_1}{L} W_1 - W_1 \qquad (11)$$

w_1 at ¼ point.

$$\frac{dM}{dx} = \tfrac{1}{4}(112.5) + \tfrac{3}{4}(0) - 0 = +$$

No maximum.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(112.5) + \tfrac{3}{4}\,(0) - 12.5 = +$$

w_2 at $\frac{1}{4}$ point.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(145) + \tfrac{3}{4}\,(0) - 12.5 = +$$

Maximum.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(145) + \tfrac{3}{4}\,(0) - 37.5 = -$$

w_3 at $\frac{1}{4}$ point.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(145) + \tfrac{3}{4}\,(12.5) - 37.5 = +$$

Maximum.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(161.25) + \tfrac{3}{4}\,(12.5) - 62.5 = -$$

w_4 at $\frac{1}{4}$ point.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(161.25) + \tfrac{3}{4}\,(12.5) - 62.5 = -$$

No maximum.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(177.5) + \tfrac{3}{4}\,(37.5) - 87.5 = -$$

Accordingly, compute the value of M by formula (10) for w_2 and w_3 at quarter point.

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \quad \ldots \ldots \ldots \ldots \quad (10)$$

M for w_2 at quarter point,

$M = \frac{1}{4}\,(2838.75) + \frac{3}{4}\,(0) - 100 = 609.7$ Kip feet.

M for w_3 at quarter point,

$M = \frac{1}{4}\,(3563.75) + \frac{3}{4}\,(37.5) - 287.5 = 631.6$ Kip feet.

The latter value, 631.6, is the maximum bending moment at the quarter point. A comparison of this value

with Table 11 shows agreement of results. Reference to Table 3 indicates that the correct wheel for maximum has been chosen.

Maximum Bending Moment at the Centre.

$$\frac{dM}{dx} = \frac{W_3 + W_1}{2} - W_2,\ (10a),\ \text{and}$$

$$M = \frac{M_3 + M_1}{2} - M_2,\ (11a),\ \text{when}\ \frac{l_1}{L} = \tfrac{1}{2}$$

w_3 at centre,

$$\frac{dM}{dx} = \frac{128.75}{2} - 37.5 = +$$

No maximum.

$$\frac{dM}{dx} = \frac{128.75}{2} - 62.5 = +$$

w_4 at centre,

$$\frac{dM}{dx} = \frac{145}{2} - 62.5 = +$$

Maximum.

$$\frac{dM}{dx} = \frac{145}{2} - 87.5 = -$$

w_5 at centre,

$$\frac{dM}{dx} = \frac{145 + 12.5}{2} - 87.5 = -$$

No maximum.

$$\frac{dM}{dx} = \frac{161.25 + 12.5}{2} - 112.5 = -$$

Therefore, maximum centre moment occurs with w_4 at centre.

$$M = \frac{2838.75}{2} - 600 = 819.37\ \text{Kip feet.}$$

This value agrees with Table 11; and the position of loading, with Table 3.

Absolute Maximum Bending Moment.

Shift w_4 according to centre of gravity rule, and then recompute the value of M under this wheel by formula (10). Note that new values for l_1, l_2, and M_3 must be determined.

By formula (13a), when w_4 is at the centre,

$$\bar{x} = \frac{M_3}{W_3} = \frac{2838.75}{145} = 19'.58$$

Therefore for absolute maximum bending moment under w_4, shift loading to left $\frac{20'.00 - 19'.58}{2} = 0'.21.$

The new values of l_1, l_2, and M_3 are

$$l_1 = 20.00 - 0.21 = 19.79$$
$$l_2 = 20.00 + 0.21 = 20.21$$
$$M_3 = 2838.75 + .21(145) = 2869.2$$

The absolute maximum bending moment =

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2$$

$$= \frac{19.79}{40} (2869.2) + 0 - 600 = 819.54 \text{ Kip feet.}$$

It appears, therefore, that the absolute maximum bending moment is .17 Kip feet greater than the maximum centre moment. The difference is not great in this particular case, as the required shift of the loading is comparatively small. The position of loading for absolute maximum bending moment agrees with Table 4, and its value agrees with Table 7.

ARTICLE V.

PIER REACTION.

In Fig. 4e is given the influence line for the pier reaction R between two non-continuous beam spans l_1 and l_2. From this influence line, the formulas (5) and (7) give

R = Ordinate-load products in $(\underline{gbh} - \underline{gak} + \underline{kzh})$

Or,

$$R = \frac{M_3}{l_2} + \frac{M_1}{l_1} - \frac{L}{l_1 l_2} M_2 = \frac{L}{l_1 l_2}\left(\frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2\right) \quad (14)$$

Formula (14) may also be derived from formula (10) since the ordinates of the influence line for R bear the constant ratio $\frac{L}{l_1 l_2}$ to the corresponding influence ordinates for M, the position of the live load and the values of l_1 and l_2 remaining fixed.

Therefore,

$$R = \frac{L}{l_1 l_2} M \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (16)$$

Substituting the value $M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2$ from formula (10) in formula (16), the result is again formula (14).

For equal spans,

$$l_1 = l_2 = l \text{ so that } R = \frac{M_3 + M_1 - 2M_2}{l} \quad . \quad (14a)$$

The rate of change of R for a movement dx of the loading to the left is

$$\frac{dR}{dx} = \frac{W_3}{l_2} + \frac{W_1}{l_1} - \frac{L}{l_1 l_2} W_2 = \frac{L}{l_1 l_2}\left(\frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2\right) \quad (15)$$

For equal spans, $l_1 = l_2 = l$, so that

$$\frac{dR}{dx} = \frac{W_3 + W_1 - 2W_2}{l} \quad . \; . \; . \; . \; . \; . \; (15a)$$

In the last member of formula (15) the quantity within the parentheses is the same as the expression for $\frac{dM}{dx}$ in formula (11). It follows, therefore, that the same position of loading gives maximum R and maximum M for any given values of l_1 and l_2.

Problem.—(*a*) Find the maximum pier reaction per rail between two simple beam spans $l_1 = 10$ ft. and $l_2 = 30$ ft. (*b*) Find the maximum pier reaction between two simple beam spans, each having a length of 20 feet. Use Cooper's $E50$ loading.

Solution of Problem (*a*).

Use formula (15) to find position of loading for maximum R.

$$\frac{dR}{dx} = \frac{L}{l_1\,l_2}\left(\frac{l}{L}\,W_3 + \frac{l_2}{L}\,W_1 - W_2\right) \quad . \; . \; (15)$$

w_2 at pier.

$$\frac{dR}{dx} = \frac{40}{10 \times 30}\left(\frac{10}{40}(145) + \frac{30}{40}(0) - 12.5\right) = +$$

Maximum.

$$\frac{dR}{dx} = \frac{40}{10 \times 30}\left(\frac{10}{40}(145) + \frac{30}{40}(0) - 37.5\right) = -$$

w_3 at pier.

$$\frac{dR}{dx} = \frac{40}{10 \times 30}\left(\frac{10}{40}(145) + \frac{30}{40}(12.5) - 37.5\right) = +$$

Maximum.

$$\frac{dR}{dx} = \frac{40}{10 \times 30}\left(\frac{10}{40}(161.25) + \frac{30}{40}(12.5) - 62.5\right) = -$$

Use formula (14) to compute the value of R.

$$R = \frac{M_3}{l_2} + \frac{M_1}{l_1} - \frac{l}{l_1 l_2} M_p.$$

w_3 at pier.

$$R = \frac{2838.75}{30} + \frac{0}{10} - \frac{40}{10 \times 30}(100) = 81^k.$$

w_4 at pier.

$$R = \frac{3563.75}{30} + \frac{37.5}{10} - \frac{40}{10 \times 30}(287.5) = 84^k.$$

The latter value of 84^k is the maximum pier reaction. Its value agrees with Table 14 and the position of loading agrees with Table 3.

Solution of Problem (b).

Use formulas (14a) and (15a),

$$R = \frac{M_3 + M_1 - 2M_2}{l}, \text{ and } \frac{dR}{dx} = \frac{W_3 + W_1 - 2W_2}{l}.$$

w_3 at pier.

$$\frac{dR}{dx} = \frac{128.75 + 0 - 2 \times 37.5}{20} = +$$

No maximum.

$$\frac{dR}{dx} = \frac{128.75 + 0 - 2 \times 62.5}{20} = +$$

w_4 at pier.

$$\frac{dR}{dx} = \frac{145 + 0 - 2 \times 62.5}{20} = +$$

Maximum.

$$\frac{dR}{dx} = \frac{145 + 0 - 2 \times 87.5}{20} = -$$

w_5 at pier.

$$\frac{dR}{dx} = \frac{145 + 12.5 - 2 \times 87.5}{20} = -$$

No maximum.

$$\frac{dR}{dx} = \frac{161.25 + 12.5 - 2 \times 112.5}{20} = -$$

Therefore, maximum pier reaction occurs when w_4 is at the pier.

$$R = \frac{2838.75 - 0 - 2 \times 600}{20} = 81.9^k.$$

This maximum pier reaction of 81.9^k agrees with value in Table 7 and Table 14, while the position of loading agrees with that given by Table 3.

ARTICLE VI.

GIRDER BRIDGE WITH PANELS.

In Fig. 6 is shown a girder bridge with panels. It is as-

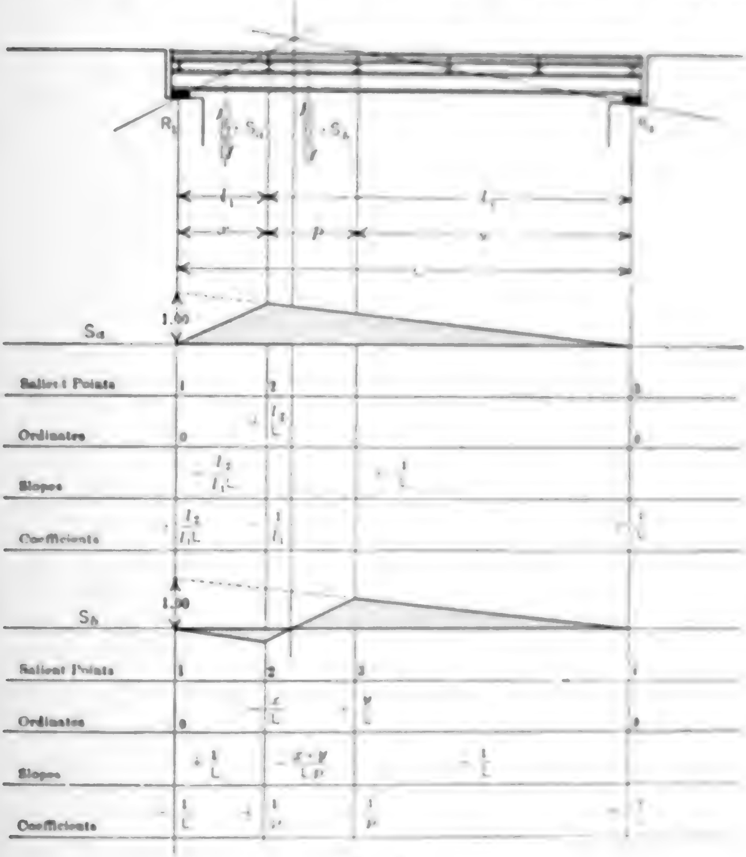

FIG. 6.

sumed that the live load has advanced beyond the left end of the span, this being the most general case.

The formulas for R_1 and R_2 are the same as formulas (9) and (9a) for the girder without panels, if the girder bridge with panels has end floor-beams; but if this bridge has end struts with the end stringers resting on separate pedestals, the value of R_1 beneath the end of the main girder is the same as S_a, the shear in the end panel, as given by formula (17) to follow.

Inasmuch as the maximum bending moment in a beam carrying concentrated loads always occurs beneath a concentration, the maximum bending moments in the main girder of a girder bridge with panels will occur at the floor-beams. The influence line for the bending moment at the floor-beams is the same as for the bending moment in a girder bridge without panels; accordingly, formulas (10) and (11) are to be used in finding maximum bending moments at the floor-beams.

It remains to derive formulas for the maximum shears S_a in the end panel and S_b in any intermediate panel. In Fig. 6 are given the influence lines for S_a and S_b. The correctness of the ordinates is at once evident. The slopes and coefficients are calculated as explained in Arts. 2 and 3. The general formulas for S_a and S_b and their rates of variation may be written at once by use of formulas (7) and (8).

$$S_a = \frac{1}{L} M_3 + \frac{l_2}{l_1 L} M_1 - \frac{1}{l_1} M_2 = \frac{1}{l_1}\left(\frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2\right) \quad (17)$$

$$\frac{dS_a}{dx} = \frac{1}{L} W_3 + \frac{l_2}{l_1 L} W_1 - \frac{1}{l_1} W_2 = \frac{1}{l_1}\left(\frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2\right) \quad (18)$$

$$S_b = \frac{1}{L} M_4 - \frac{1}{p} M_3 + \frac{1}{p} M_2 - \frac{1}{L} M_1 \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (19)$$

$$\frac{dS_b}{dx} = \frac{1}{L} W_4 - \frac{1}{p} W_3 + \frac{1}{p} W_2 - \frac{1}{L} W_1 \quad . \; . \; . \; . \; . \; . \; . \quad (20)$$

Formula (17) when compared with formula (10) shows that S_a is equal to the bending moment at the first intermediate floor-beam divided by the length of the first panel. Formula (18) when compared with formula (11) shows that

the same position of loading that gives maximum bending moment at the first intermediate floor-beam will also give maximum shear in the end panel.

Formulas (19) and (20) are perfectly general and will serve for any assumed series of vertical loads in any position. For the usual standard loadings and panel lengths, however, it is not necessary to advance any loads beyond an intermediate panel for maximum shear in this panel. Therefore, for practical purposes formulas (19a) and (20a)

$$S_b = \frac{M_4}{L} - \frac{M_3}{p} = \frac{1}{p}\left(\frac{p}{L}M_4 - M_3\right) \quad . \quad . \quad (19a)$$

$$\frac{dS_b}{dx} = \frac{W_4}{L} - \frac{W_3}{p} = \frac{1}{p}\left(\frac{p}{L}W_4 - W_3\right) \quad . \quad (20a)$$

Illustrative Problem.—A single track through girder bridge with a floor system consisting of stringers and floor-beams, both end and intermediate, has six panels of 20 feet each. Find the maximum end reaction and the shear in panels 0 – 1, 1 – 2, and 2 – 3, using Cooper's $E50$ loading.

Solution.—For maximum end reaction place wheel 2 at left end. Use formula

$$R_1 = \frac{M_2 - M_1}{L} - W_1 \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$R_1 = \frac{27651 - 100}{120} - 12.5 = 217.1^k$$

Note that the above value agrees with Table 7.

For maximum shear in panel 0 – 1, find critical wheel by formula (18) and then compute shear by formula (17).

Try wheel 3 at panel point 1.

$$\frac{dS_a}{dx} = \frac{1}{20}\left(\frac{1}{6}(365) + 0 - 37.5\right) = +$$

Maximum.

$$\frac{dS_a}{dx} = \frac{1}{20}\left(\frac{1}{6}(365) - 0 - 62.5\right) = -$$

Note that the position of loading agrees with Table 3. For this position of loading formula (17) gives

$$S_a = \frac{1}{20}\left(\frac{1}{6}(21895) + 0 - 287.5\right) = 168.1^k.$$

For maximum shears in the intermediate panels, determine the position of loading by formula (20a) and the shear by formula (19a).

$$\frac{dS_b}{dx} = \frac{1}{p}\left(\frac{p}{L} W_4 - W_3\right) \quad \text{. . . . . (20a)}$$

$$S_b = \frac{1}{p}\left(\frac{p}{L} M_4 - M_3\right) \quad \text{. . . . . (19a)}$$

Panel 1–2. Try wheel 3 at panel point 2.

$$\frac{dS_b}{dx} = \frac{1}{20}\left(\frac{1}{6}(306.25) - 37.5\right) = +$$

Maximum.

$$\frac{dS_b}{dx} = \frac{1}{20}\left(\frac{1}{6}(322.50) - 62.5\right) = -$$

$$S_b = \frac{1}{20}\left(\frac{1}{6}(15051.25) - 287.5\right) = 111.0^k.$$

Panel 2–3. Try wheel 3 at panel point 3.

$$\frac{dS_b}{dx} = \frac{1}{20}\left(\frac{1}{6}(240) - 37.5\right) = +$$

Maximum.

$$\frac{dS_b}{dx} = \frac{1}{20}\left(\frac{1}{6}(240) - 62.5\right) = -$$

$$S_b = \frac{1}{20}\left(\frac{1}{6}(9345) - 287.5\right) = 63.5^k.$$

The above values for shears agree with the values given by Table 9. The wheel for maximum shear in panels of girder and truss bridges is given in Table 6.

ARTICLE VII.

THROUGH PRATT TRUSS. GENERAL FORMULAS FOR LIVE-LOAD STRESSES AND THEIR RATE OF VARIATION. ILLUSTRATIVE PROBLEMS.

THE general formulas $S = \Sigma CM$ and $\dfrac{dS}{dx} = \Sigma CW$ may be used to write the equations for the live-load stresses in any member of a framed structure as soon as its influence

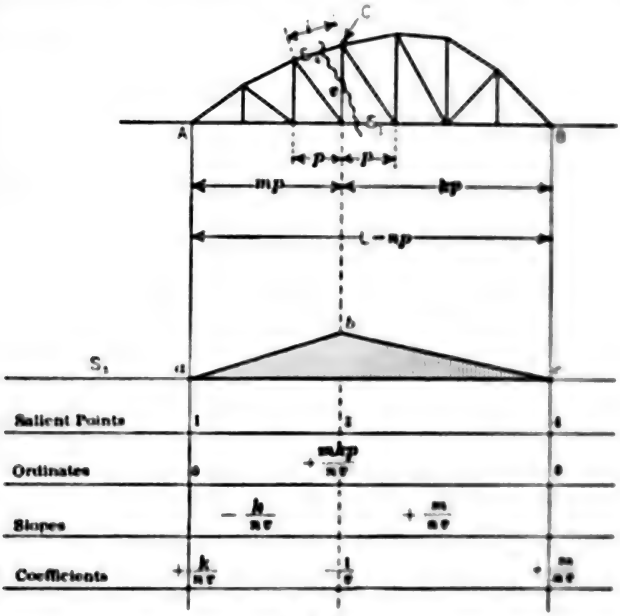

FIG. 7.

line has been drawn and the ordinates at the salient points determined.

In Figs. 7, 8, 9, and 10 are shown all the influence lines

needed in writing the formulas for the live-load stresses in a through Pratt truss with non-parallel or parallel chords. The influence ordinate at any salient point is the calculated stress due to a one-pound load on the bridge at the panel point above this salient point. By easily discovered relations between similar triangles, the algebraic value of each stress, or influence ordinate, is expressed in terms that are most readily evaluated in any numerical problem.

The derivation of any one formula for a live-load stress is typical. Refer to Fig. 7. The stress in the lower chord member S_5 is found by taking moments about C. The influence line for S_5 is straight over each of the two intervals kp and mp. The ordinates at the salient points 1 and 4 are zero. The ordinate at salient point 3 must be found by placing a one-pound load at the lower panel point of the truss above this salient point and calculating the value of S_5. For the unit load so placed,

$$\text{Reaction at } A = \frac{kp}{np} = \frac{k}{n}$$

By moments about C,

$$\frac{k}{n}(mp) = S_5\,(v)$$

Therefore,

$$S_5 = +\frac{mkp}{nv} = \text{Influence ordinate at 3.}$$

The slopes of the segments of this influence line follow.

$$\text{Slope of } ab = -\frac{mkp}{nv} \div mp = -\frac{k}{nv}$$

$$\text{Slope of } bc = +\frac{mkp}{nv} \div kp = +\frac{m}{nv}$$

The coefficients C for use in the general formula $S = \Sigma CM$ are now found.

$$C_1 = 0 + \frac{k}{nv} = +\frac{k}{nv}$$

$$C_3 = -\frac{k}{nv} - \frac{m}{nv} = -\frac{1}{v}$$

$$C_4 = \frac{m}{nv} - 0 = +\frac{m}{nv}$$

Therefore, for the position of the live load advanced beyond the limits of the span, the general formula for S_3 is

$$S_3 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 + \left(\frac{k}{nv}\right)M_1.$$

However, in actual practice it is usually not necessary to advance the loading beyond the left end of the span in order to get a maximum value of S_3. The usual formula will therefore not contain the term M_1, since this will be zero; thus,

$$S_3 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \quad . \quad . \quad . \quad (21)$$

Inasmuch as the horizontal component of the stress S_4 in an inclined top chord member or end post equals the stress S_3 in a corresponding lower chord member, the stress S_4 in any top chord member or end post may be found by

$$S_4 = \frac{i}{p} \cdot S_3 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

In Fig. 8 is shown the influence line for the stress S_5 in any vertical post. The influence ordinates are determined by taking moments about the intersection of the upper and lower chord members which are cut by the section. The algebraic values of these ordinates are transformed by use of easily discovered relations between similar triangles. The slopes and coefficients are then calculated in the usual way. The influence line indicates that the live load should advance into but not beyond the panel p for a maximum compression, and for this reason M_1 and M_2 equal zero for the usual case. The numerical value of

the maximum *compression* S_4 in a vertical post is, therefore,

$$S_4 = \left(\frac{a}{bL}\right)M_4 - \left(\frac{1}{p}\right)M_3 \quad . \ . \ . \ . \ . \ . \quad (23)$$

The coefficients for the stress in any inclined web member are given by Fig. 9. The quantities for S_1 and S_2 are

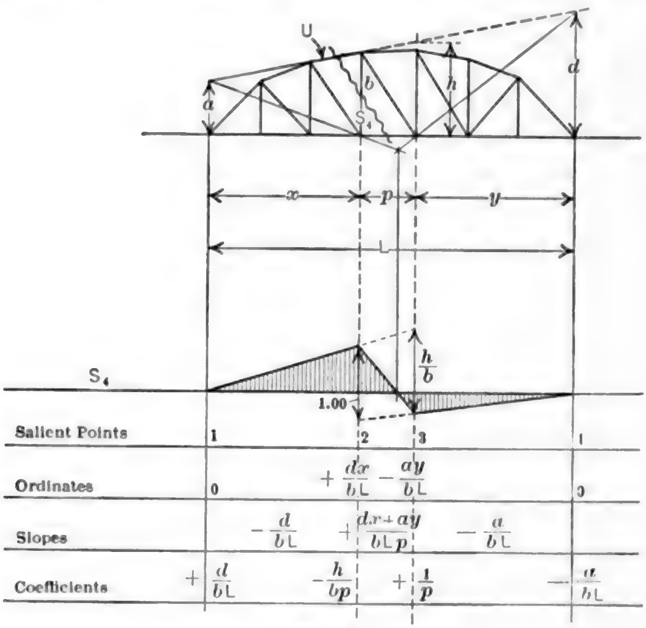

FIG. 8.

as shown, and the quantities for S_3 are of the same algebraic form except that they are of opposite sign throughout. For the usual position of the live load advanced from the right into but not beyond the panel p for maximum stress, the moment sums M_1 and M_2 equal zero, and the numerical values of the maximum tension S_1 and S_2 and of the maximum compression S_3 are given by the following formula:

$$S_1,\ S_2,\text{ or }S_3 = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_3 \quad . \ . \ . \quad (24)$$

In a special case where the loading must be advanced beyond the panel p until the tension in the inclined counter-web member S_2 is balanced by the dead-load compression

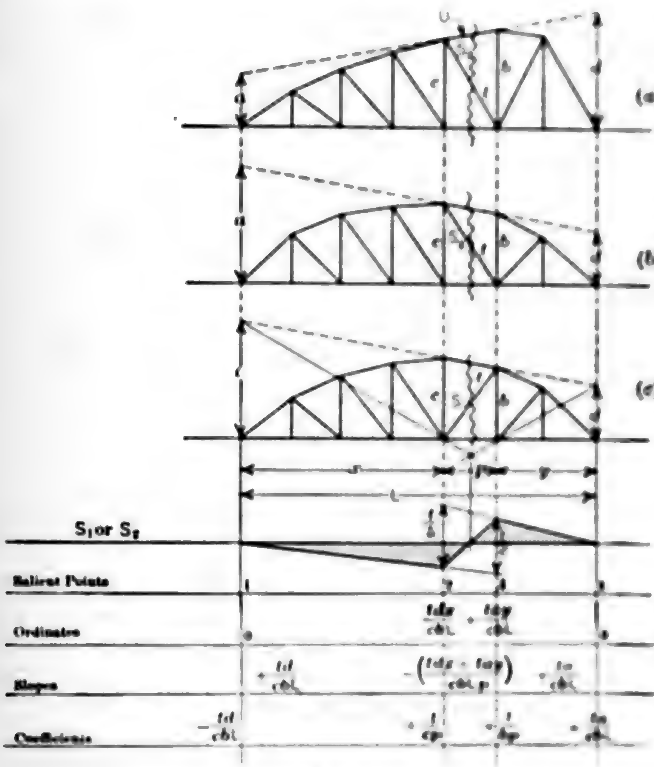

FIG. 9.

in this same member, the value of M_3 is not zero, and the formula for S_2 becomes

$$S_2 = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_3 + \left(\frac{t}{cp}\right)M_2$$

Or, letting $M_e = \left(M_3 - \frac{b}{c}M_2\right)$,

$$S_2 = \left(\frac{ta}{cbL}\right)M_4 - \frac{t}{bp}\left(M_3 - \frac{b}{c}M_2\right) = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_e \quad (2$$

Note that the coefficients of M_4 and M_c in this formula are the same as the coefficients for M_4 and M_3 in formula (24).

The influence line for the counter-tension in a vertical post is shown in Fig. 10. For the usual case, the loading advances beyond the panel but not beyond the end of the span. Therefore M_1 is equal to zero, so that

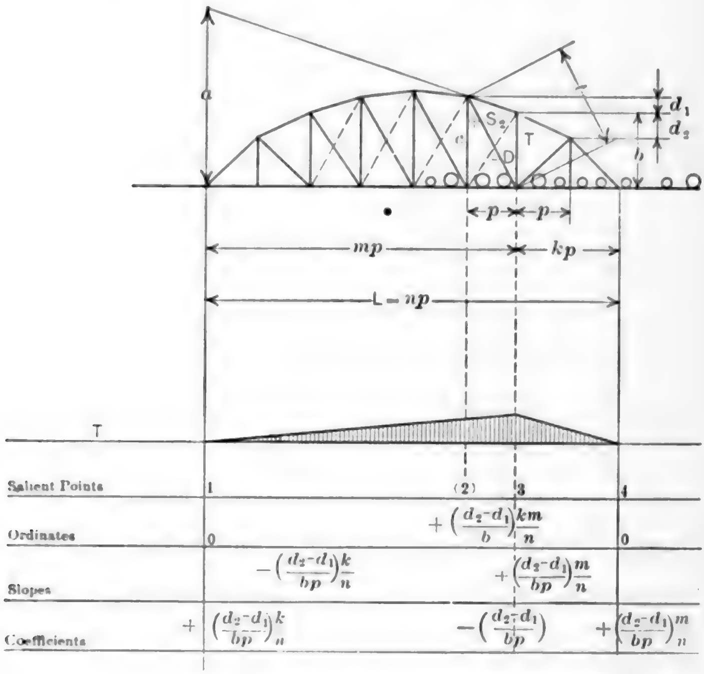

FIG. 10.

$$T = \left(\frac{d_2 - d_1}{bp}\right)\left(\frac{m}{n} M_4 - M_3\right) = K \cdot M_o \quad . \quad . \quad (26)$$

where K and M_0 stand for the corresponding terms in the parentheses. In order that T be a maximum the live load must advance beyond the position for the maximum tension S_2 until the tension as computed by formula (25) becomes equal to the dead-load compression in this same member. For this position of the live load, the value of T is then computed by using formula (26). It may be noted that

some specifications state that only $\frac{2}{3}$ of the dead-load compression is to be counted as effective in counteracting the live-load tension in an inclined counter-web member. This specification has been observed in the problem to follow.

A review of the preceding formulas shows that all the live-load stresses may be computed by formulas (21), (22), (23), and (24), except the counter-tension in a vertical post and the tension in a floor-beam hanger. Formula (25) makes it possible to find readily by trial the position of loading for maximum counter-tension in a vertical post, and formula (26) gives the value of this tension. The maximum tension in the floor-beam hanger may be found by the use of formulas (14a) and (15a) for pier reaction between equal spans.

If the chords of the Pratt truss are parallel, there will be no counter-tension in any vertical post. Formula (21) for the stress in a horizontal chord member and formula (22) for the stress in the inclined end post remain unchanged. Formulas (23) and (24) for web stresses are simplified because $a = b =$ depth of truss.

The formulas, therefore, for the Pratt truss with parallel chords are:

Stress in horizontal chord members =

$$S_3 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{r}\right)M_3 \quad \ldots \quad (21)$$

Stress in inclined end post $= S_4 = \frac{i}{p} S_3 \quad \ldots \quad (22)$

Stress in vertical post $= S_2 = \left(\frac{1}{L}\right)M_4 - \left(\frac{1}{p}\right)M_3 \quad \ldots \quad (29)$

Stress in inclined web member =

$$S_1 = \left(\frac{t}{cL}\right)M_4 - \left(\frac{t}{cp}\right)M_3 = \frac{t}{c} S_2 \quad \ldots \quad (30)$$

One general formula will suffice for finding the position of loading for maximum chord and web stresses of a Pratt truss with either inclined or parallel chords. The formulas

(21), (23), (24), (29), and (30) for these stresses are of one general form

$$S = (G)\, M_4 - (H)\, M_3 \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

where G and H are the corresponding coefficients of M_4

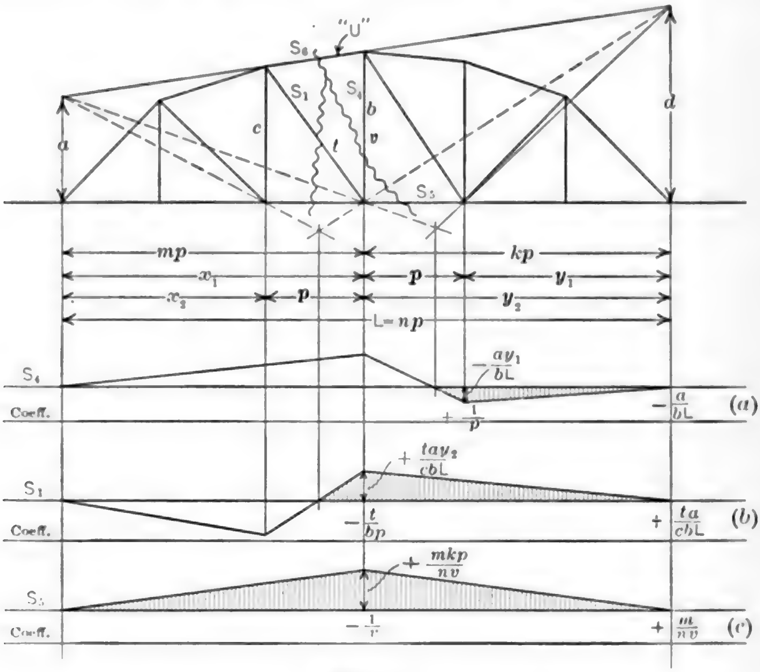

FIG. 11.

and M_3 in the preceding formulas. The rate of variation of S as the load advances is

$$\frac{dS}{dx} = GW_4 - HW_3 = H\left(\frac{G}{H}\, W_4 - W_3\right) \quad . \quad . \quad (28)$$

When any one of the above stresses is a maximum, the value of $\left(\frac{G}{H}\, W_4 - W_3\right)$ passes through zero as a wheel is shifted from right to left of the salient point 3 in Figs. 7, 8, or 9.

The preceding formulas for the live-load stresses are summarized for convenient reference in Art. 11 preceding

the Tables. The important dimensions and quantities in Figs. 7, 8, and 9 are summarized in Fig. 11. If a uniform live load is used, the shaded areas in Fig. 11a, b and c multiplied by the intensity of the uniform load will give the maximum live-load stresses. The algebraic value of any one of these triangular areas is conveniently expressed as the base of the triangle times ½ of the given algebraic ordinate. The lengths of the bases of the shaded areas in Figs. 11a and b may be readily determined by one of the constructions shown in Figs. 12a and 12b, which give the position of the unit load for zero stress in the members indicated. The proofs that these constructions locate neutral points are not given, for they are generally known, and are proved in numerous texts on bridges. (See Marburg's "Framed Structures and Girders," Vol. I, page 392.)

The application of the preceding formulas will now be made to the calculation of the live-load stresses in the two single track through Pratt trusses shown in Figs. 13 and 14. A convenient procedure is as follows:

1. Determine the lengths of all inclined members and write their values on the truss outline.

2. Determine the values of the intercepts *a* as defined by Fig. 11 and write their values on the truss outline.

3. Write on the truss outline the distances of the several panel points from the right end of the span.

4. Write down the reciprocals of the span, panel length, and lengths of vertical members.

5. Make a form for tabulating calculations and list members in some convenient form as is done in Figs. 13 and 14.

6. Calculate the numerical values of the coefficients G and H for the several members by use of the formulas already derived.

7. Determine the position of the loading for maximum stress by finding the position of loading causing $\left(\frac{G}{H}W_s - W_s\right)$

to pass through zero, and for this position of loading select from Table 2 the corresponding values of M_4 and M_3. At

VARIOUS CONSTRUCTIONS USED TO FIND NEUTRAL POINTS IN PRATT TRUSSES.

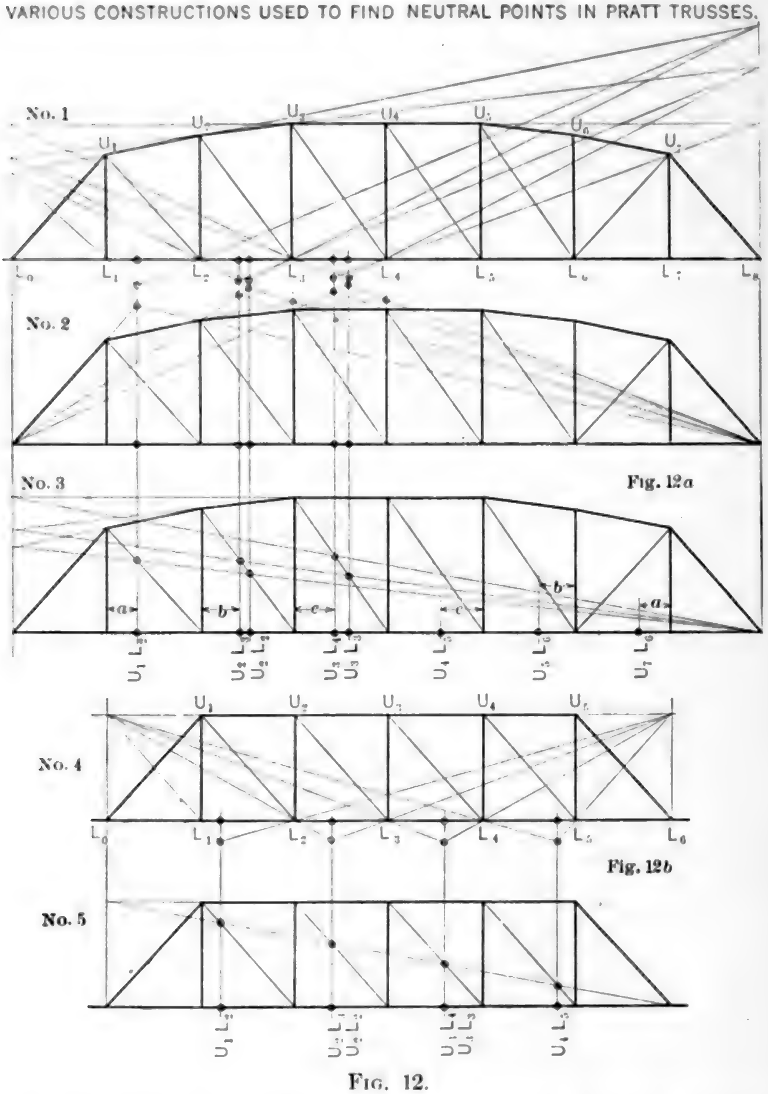

FIG. 12.

the same time tabulate the length L_1 of loading causing maximum stress as this value is used in the impact formula

$$I = S \cdot \frac{300}{L_1 + 300}.$$

8. Calculate values of $S = GM_4 - HM_5$ and combine with impact and dead-load stresses. When the dead- and live-load stresses are of opposite sign, the combination is usually not algebraic but according to the particular specification that is used.

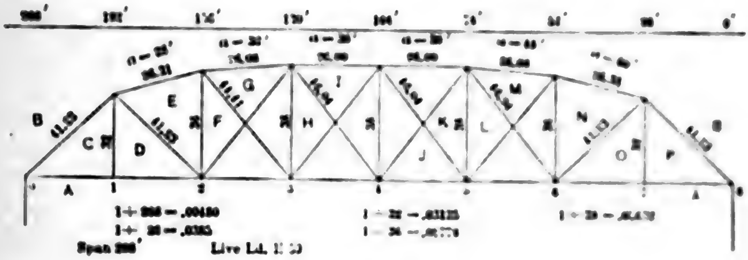

FIG. 13.

Mem.	G	H	Wheel	M_4	M_5	GM_4	HM_5	S	L_1	$\frac{300}{L_1+300}$	I	D.L.	Total K
EF	.00373	.0385	3 @ 3	33970	287	127	11	−116	143	.677	− 78	− 40	−234
ED	.00481	.0442	3 @ 2	46255	287	223	13	+210	169	.640	+134	+ 83	+427
GH	.00405	.0385	2 @ 4	21531	100	87	4	− 83	112	.728	− 60	− 15	−158
GF	.00500	.0450	3 @ 3	33970	287	170	13	+157	143	.677	+106	+ 48	+311
IJ	.00480	.0385	2 @ 5	12940	100	62	4	− 58	86	.777	− 45	+ 7	......
IH	.00580	.0466	3 @ 4	23375	287	136	13	+123	117	.719	+ 88	+ 21	+232
JK	.00580	.0466	2 @ 5	12940	100	75	5	+ 70	86	.777	+ 54	− 21	......
ML	.00777	.0493	2 @ 6	6550	100	51	5	+ 46	60	.833	+ 38	− 50	......
NO	.01030	.0496	2 @ 7	2307	100	24	5	− 19	34	.898	− 17	+ 83	No counter
AC = AD	.00390	.0312	4 @ 1	63111	600	247	19	+228	200	.600	+137	+101	+466
BC	......	......	......	......	......			−362			−217	−160	−739
AF	.00695	.0278	7 @ 2	59095	2694	410	75	+335	193	.608	+203	+154	+692
BE	......	......	......	......	......			−339			−206	−156	−701
AH	.00985	.0263	11 @ 3	59661	7310	587	192	+395	194	.607	+239	+181	+815
BG	......	......	......	......	......			−396			+240	−181	−817
BI	.01315	.0263	13 @ 4	50901	9585	670	252	−418	178	.627	−262	−194	−874
CD	.0385	.0770	4 @ 1	3725	600	144	46	+ 98	44	.872	+ 86	+ 25	+209

Post at	Mem.	M.	Mc	S	½ D	K	M.	T	L_1	$\frac{300}{L_1+300}$	I	D.L.	Total
5	JK	22261	2390	+16	−14	.00203	11340	+23	114	.725	+17	+3	+ 43
6	ML	8865	687	+35	−34	.00214	5960	+13	71	.8	+10	+1	+ 24

9. Find positions of loading for maximum counter-tensions in posts and compute values by use of formulas (25) and (26).

Problem 1.

Calculation of Live-load Stresses in a Pratt Truss with Inclined Chord.

The complete data for this problem are given in Fig. 13. Items 1 to 5 of the above method of procedure need no explanation. The values of the coefficients G and H, the position of the loading for maximum stress, and the value of the maximum stress will be determined for several typical members; for example, vertical post, inclined web members, horizontal chords, end post, and inclined chords.

Vertical Post EF.

Formula $$S_4 = \left(\frac{a}{bL}\right) M_4 - \left(\frac{1}{p}\right) M_3 \quad . \quad . \quad . \quad . \quad . \quad . \qquad (23)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{a}{bL} = \frac{28}{36}\,(.00480) = .00373$$

$$H = \frac{1}{p} = .0385$$

Try w_3 at panel point 3. Use Table 2. $L_1 = 143'$.

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00373}{.03850}\,(440.0) - \begin{matrix} 37.5 \\ \text{or} \\ 62.5 \end{matrix} = \begin{matrix} + \\ \text{or} \\ - \end{matrix}$$

Therefore w_3 at 3 gives a maximum.

$$S = GM_4 - HM_3 = .00373(33970) - .0385(287.5)$$
$$= 126.7 - 11.0 = 115.7^k$$

$$\text{Impact factor} = \frac{300}{L_1 + 300} = \frac{300}{443} = .677$$

$$\text{Impact stress} = .677 \times 115.7 = 78.3^k.$$

Inclined Web Member ED.

Formula $$S_1 = \left(\frac{ta}{cbL}\right) M_4 - \left(\frac{t}{bp}\right) M_3 \qquad (24)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{ta}{cbL} = \frac{41.23 \times 28}{32 \times 36}(.00480) = .00481$$

$$H = \frac{t}{bp} = \frac{41.23}{36}(.0385) = .0442$$

Try w_3 at panel point 2. Use Table 2. $L_1 = 160'$.

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00481}{.0442}(505.0) - \begin{matrix} 37.5 \\ \text{or} \\ 62.5 \end{matrix} = \begin{matrix} + \\ \text{or} \\ - \end{matrix}$$

Therefore w_3 at 2 gives a maximum.

$$\therefore S = GM_4 - HM_3 = .00481(46255) - .0442(287.5)$$
$$= 223 - 13 = 210^k.$$

$$\text{Impact factor} = \frac{300}{469} = .640$$

$$\text{Impact stress} = .640 \times 210 = 134^k.$$

Inclined Web Member ML.

Formula $$S_2 = \left(\frac{ta}{cbL}\right) M_4 - \left(\frac{t}{bp}\right) M_3 \qquad (24)$$

Refer to Fig. 9 or Fig. 11 for definition of dimensions.

$$G = \frac{ta}{cbL} = \frac{46.04 \times 48}{38 \times 36}(.00480) = .00777$$

$$H = \frac{t}{bp} = \frac{46.04}{36}(.0385) = .0493$$

Try w_2 at panel point 6. Use Table 2. $L_1 = 60'$.

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00777}{.0493}(190) - \begin{matrix} 12.5 \\ \text{or} \\ 37.5 \end{matrix} = \begin{matrix} + \\ \text{or} \\ - \end{matrix}$$

Therefore w_2 at 6 gives a maximum.

$$S = GM_4 - HM_3 = .00777(6550) - .0493(100)$$
$$= 51 - 5 = 46^k.$$

$$\text{Impact factor} = \frac{300}{360} = .833$$

$$\text{Impact stress} = .833 \times 46 = 38^k.$$

Lower Chord Member AC = AD.

Formula $$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \quad . \quad . \quad . \quad . \quad (21)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{m}{nv} = \tfrac{1}{8}\,(.03125) = .00390$$

$$H = \frac{1}{v} = .0312$$

Try w_4 at panel point 1. Use Table 2. $L_1 = 200'$.

$$\left(\frac{G}{H}W_4 - W_3\right) = \frac{.00390}{.0312}(582.5) - \begin{matrix}62.5\\ \text{or}\\ 87.5\end{matrix} = \begin{matrix}+\\ \text{or}\\ -\end{matrix}$$

Therefore w_4 at 1 gives a maximum.

$$S = GM_4 - HM_3 = .00390(63111) - .0312(600)$$
$$= 247 - 19 = 228^k.$$

$$\text{Impact factor} = \frac{300}{500} = .600$$

$$\text{Impact stress} = .600 \times 228 = 137^k.$$

End of Post BC.

Formula $$S_6 = \frac{i}{p}\,S_5 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

$$S_6 = \frac{41.23}{26}(228) = 362^k, \text{ and impact } = \frac{41.23}{26}(137) = 217^k.$$

Lower Chord Member AH.

Formula $$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \quad . \quad . \quad . \quad (21)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{m}{nv} = \tfrac{3}{8}\,(.02632) = .00985$$

$$H = \frac{1}{v} = .0263$$

Try w_{11} at panel point 3. Use Table 2. $L_1 = 194'$.

$$\left(\frac{G}{H}W_4 - W_3\right) = \frac{.00985}{.0263}(567.5) - \begin{matrix}190\\ \text{or}\\ 215\end{matrix} = \begin{matrix}+\\ \text{or}\\ -\end{matrix}$$

Therefore w_{11} at 3 gives a maximum.

$$S = GM_4 - HM_3 = .00985(59661) - .0263(7310)$$

$$= 587 - 192 = 395^k.$$

$$\text{Impact stress} = \frac{300}{494}S = .607 \times 395 = 239^k.$$

Top Chord Member BG.

Formula

$$S_6 = \frac{i}{p}S_5 \qquad \text{. . . . .} \qquad (22)$$

$$S_6 = \frac{26.08}{26}(395) = 396^k.$$

$$\text{Impact} = \frac{26.08}{26}(239) = 240^k.$$

Counter-Tension in Post at Panel Point 5.

Formulas

$$S_2 = \text{Stress } JK = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)\left(M_3 - \frac{b}{c}M_2\right)$$

$$= \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_e \qquad \text{. . . . . .} \qquad (25)$$

T = tension in post.

$$= \left(\frac{d_2 - d_1}{bp}\right)\left(\frac{m}{n}M_4 - M_3\right) = K \cdot M_o \qquad (26)$$

Refer to Fig. 10 for definition of dimensions.

The calculation of the dead-load compression in JK is

not given, but the value is 21^k. Two-thirds of this compression, or 14^k, will be considered effective in counterbalancing the live-load tension in JK. The live load must be advanced beyond the position of maximum live-load tension in JK (*i.e.*, w_2 at panel point 5) until S_2, or the stress in JK, equals 14^k. This must be done by trial, S_2 being figured each time by formula (25). It is found that when 114′ of loading has advanced upon the bridge, this condition is approximately satisfied. For this position of loading

$$M_4 = 22261$$

$$M_c = \left(M_3 - \frac{b}{c}M_2\right) = (2565 - 175) = 2390$$

$$G = \left(\frac{ta}{cbL}\right) = \frac{46.04 \times 38}{38 \times 38}(.00480) = .00580$$

$$H = \left(\frac{t}{bp}\right) = \frac{46.04}{38}(.0385) = .0466$$

Therefore,

$$S_2 = .00580(22261) - .0466(2390) = 16^k.$$

This value of $S_2 = 16^k$ balances $\frac{2}{3}\ D = -\ 14^k$, nearly enough for practical purposes. Therefore, compute T for this position of the live load.

$$T = \left(\frac{d_2 - d_1}{bp}\right)\left(\frac{m}{n}M_4 - M_3\right) = K \cdot M_o$$

$$K = \frac{2 - 0}{38 \times 26} = .00203$$

$$M_o = \tfrac{5}{8}(22261) - 2565 = 11340$$

$$T = .00203(11340) = 23^k$$

$$\text{Impact factor} = \frac{300}{414} = .725$$

$$\text{Impact stress for } T = .725 \times 23 = 17^k.$$

Problem 2.

Live-load Stresses in a Pratt Truss with Parallel Chords.

The complete data for this problem are given in Fig. 14. Formulas (21), (29), and (30) give the values of the

coefficients G and H, which are identical for several members of any Pratt truss with parallel chords. The procedure for finding the positions of the loading and maximum stresses is exactly as in Problem 1. It should be noted that

$$\text{Stress } FG = \text{Stress } EF \times \frac{37.54}{28}$$

$$\text{''} \quad HI = \text{''} \quad GH \times \frac{37.54}{28}$$

$$\text{''} \quad BC = \text{''} \quad AC \times \frac{37.54}{25}$$

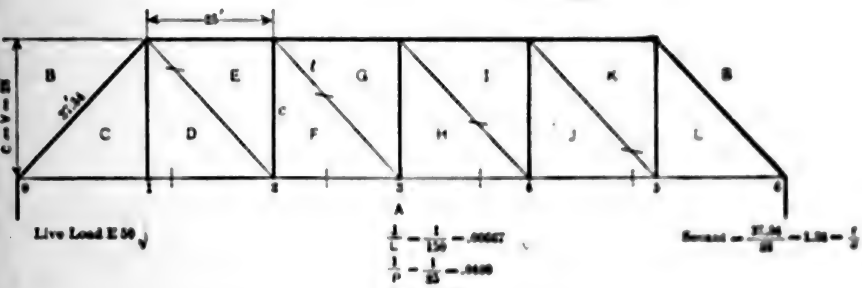

Fig. 14.

Mem.	G	H	Wheel	M_1	M_2	S
CD	.0400	.0800	4 @ 1	3564	600	95
EF	.00667	.0400	3 " 3	13520	287	79
FG	......	......	......	......	....	106
GH	.00667	.0400	2 " 4	6170	100	37
HI	......	......	......	......	....	50
JK	.00894	.0536	2 " 5	2179	100	14
DE	.00894	.0536	3 " 2	21895	287	181
BC	......	......	......	......	....	272
AC = AD	.00595	.0357	4 " 1	33970	600	181
AF = BE	.01190	.0357	7 " 2	31375	2694	278
BG	.01785	.0357	12 " 3	34411	8385	314

The stresses in all of the chord members may be checked by use of Table 8, and the stresses in the end post and web members may be checked by Table 9. The stress in CD agrees with the maximum pier reaction in Table 7. Table 3 may be used to find the position of loading for maximum chord stresses, and Table 6 gives position of loading for maximum web stresses.

ARTICLE VIII.

THREE-HINGED ARCH. APPLICATION OF THE GENERAL METHOD TO THE CALCULATION OF LIVE-LOAD STRESSES.

THE general formulas $\dfrac{dS}{dx} = \Sigma CW$ and $S = \Sigma CM$ may be used directly to find the position of loading and the

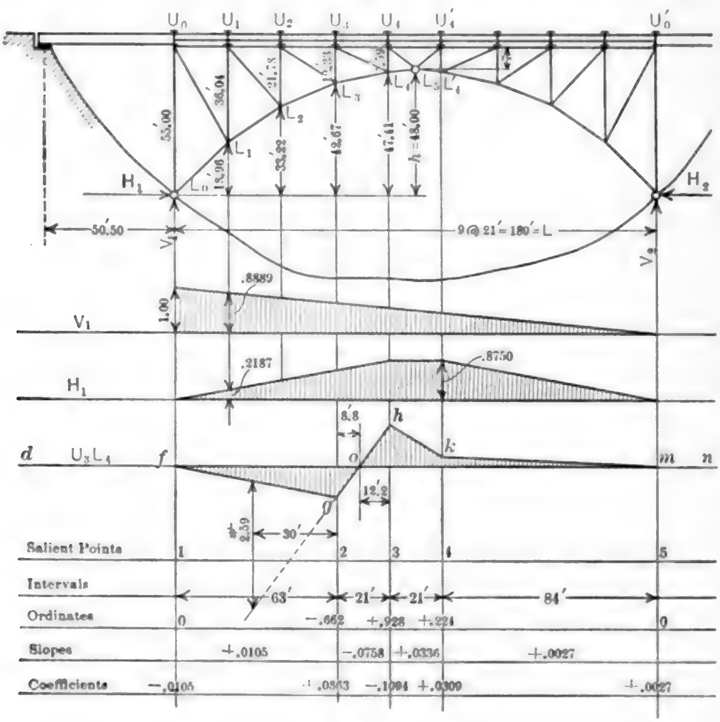

FIG. 15.

value of the maximum live-load stress in any member of a framed structure as soon as the influence line for this member and the ordinates at all salient points have been

determined. This method is applied to the calculation of maximum live-load stresses for the three-hinged arch shown in Fig. 15. Cooper's $E40$ loading is used.

First are drawn the influence lines for the horizontal and vertical components of the reaction at the left hinge. The vertical component V_1 is the same as for a simple span L. The horizontal component H_1 equals the bending moment at the centre of the span L divided by the depth h. The influence-line ordinates for all members are now found by drawing five Maxwell diagrams, one of which is reproduced in Fig. 16. From the influence lines for V_1 and H_1, the value of V_1 is .8889 and H_1 is .2187 for a one-pound load at U_1. The external loads acting on the left half of the arch are then as shown in Fig. 16a. The load line *axbcya* in Fig. 16b is drawn to a scale of 10″ = 1 pound, and the Maxwell diagram completed in the usual way. The scaled

TABLE A

INFLUENCE-LINE ORDINATES FOR THREE-HINGED ARCH

Members	Ordinates				
	1 lb. at U_1	1 lb. at U_2	1 lb. at U_3	1 lb. at U_4	1 lb. at U'_3
U_0U_1 = ...	− .403	− .223	− .045	+ .130	+ .201
U_1U_2 = ...	− .417	− .833	− .286	+ .262	+ .477
U_2U_3 = ...	− .378	− .756	−1.135	+ .189	+ .757
U_3U_4 = ...	− .171	− .342	− .513	− .685	+ .548
L_0L_1 = ...	− .295	− .590	− .885	−1.180	−1.182
L_1L_2 = ...	+ .221	− .264	− .740	−1.224	−1.302
L_2L_3 = ...	+ .217	+ .434	− .408	−1.248	−1.484
L_3L_4 = ...	+ .164	+ .328	+ .491	−1.086	−1.674
L_4L_5 = ...	− .048	− .096	− .145	− .193	−1.420
U_0L_0 = ...	− .692	− .384	− .075	+ .234	+ .345
U_1L_1 = ...	−1.014	− .632	− .253	+ .129	+ .287
U_2L_2 = ...	+ .022	− .955	− .490	− .043	+ .165
U_3L_3 = ...	+ .075	+ .150	− .775	− .317	− .076
U_4L_4 = ...	+ .114	+ .226	+ .342	− .545	− .364
U_0L_1 = ...	+ .800	+ .441	+ .085	− .270	− .400
U_1L_2 = ...	+ .019	+ .878	+ .350	− .180	− .398
U_2L_3 = ...	− .044	− .088	+ .986	+ .086	− .324
U_3L_4 = ...	− .221	− .442	− .662	+ .928	+ .224
U_4L_5 = ...	− .206	− .412	− .617	− .823	+ .657
H	0.2187	0.4375	0.6562	0.8750	0.8750
V	0.8889	0.7777	0.6666	0.5555	0.4444
θ	14°	29°	44°	58°	63°

values of these stresses are the influence ordinates for a one pound load at U_1. In an exactly similar way the influence ordinates for a unit load at U_2, U_3, U_4, and U'_4 are determined. The influence lines are straight from U'_0 to

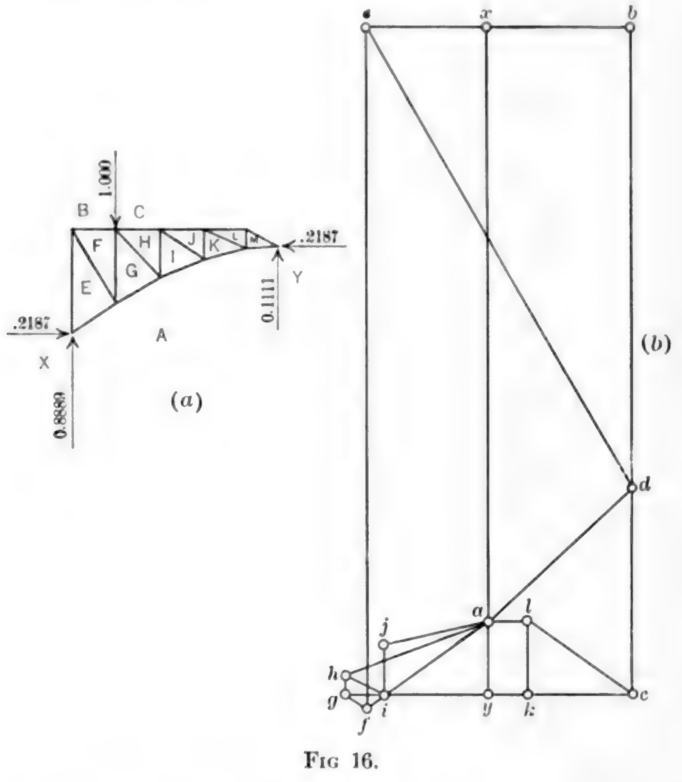

FIG 16.

U'_4. Table A gives the influence ordinates for all members and also for the horizontal and vertical components of the reaction at the left hinge. The angle θ is the inclination of this reaction with the vertical.

The calculation of the live-load stresses in any one member is typical. The member U_3L_4 is taken. The influence line for this member is drawn to scale in Fig. 15 by use of the influence ordinates from Table A. The salient points occur below panel points U_3, U_4, and U'_4. The distance

from U_3 to the neutral point 0 equals $\frac{.662}{.662 + .928}(21) = 8'.8$.

Calculation of Slopes.

Slope of $df = 0$

$$fg = \frac{0 - (-.662)}{68} = +.0105$$

$$gh = \frac{-.662 - (.928)}{21} = -.0758$$

$$hk = \frac{.928 - (.224)}{21} = +.0336$$

$$km = \frac{.224 - 0}{84} = +.0027$$

$$mn = 0$$

Calculation of Coefficients.

$$\begin{aligned}
C_1 &= \quad 0 - (.0105) &&= -.0105 \\
C_2 &= \quad .0105 - (-.0758) &&= +.0863 \\
C_3 &= -.0758 - (.0336) &&= -.1094 \\
C_4 &= \quad .0336 - (.0027) &&= +.0309 \\
C_5 &= \quad .0027 - \quad 0 &&= +.0027
\end{aligned}$$

The sum of these coefficients equals zero. This agrees with formula (6) of Art. 3.

It should be remembered, as is pointed out in Art. 3, that the value of these coefficients may be measured graphically. For example, in Fig. 15 the value of C_2 is $\frac{2.59}{30} =$ .0863.

By use of the formula $\frac{dS}{dx} = \Sigma CW$ and Rule 1 of Art. 3, the position of loading for maximum tension in U_3L_4 may now be determined. Try wheel 3 at U_4 with the loading advancing toward the left. Take the values of the load sums and moment sums for $E40$ from Table 2.

$$\frac{dS}{dx} = \Sigma CW = -.1094(30) + .309(103) + .0027(302) = +.7$$

$$\frac{dS}{dx} = \Sigma CW = -.1094(50) + .309(103) + .0027(302) = -.7$$

Therefore w_3 at U_4 gives a maximum tension in U_3L_4, and its value is

$$S = \Sigma CM = -.1094(230) + .309(1846) + .0027(19001) = 83^k.$$

By use of the formula $\frac{dS}{dx} = \Sigma CW$ and Rule 2 of Art. 3, the position of loading for maximum compression in U_3L_4 is now determined. Try wheel 2 at U_3 with the loading advancing toward the right. Note that the signs of the coefficients remain unchanged. Take the values of the load sums and moment sums for $E40$ from Table 2.

$$\frac{dS}{dx} = \Sigma CW = -.0105(192) + .0863(10) = -1.3$$

$$\frac{dS}{dx} = \Sigma CW = -.0105(192) + .0863(30) = +0.6$$

Therefore w_2 at U_3 gives a maximum negative stress, or compression, in U_3L_4, and its value is

$$S = \Sigma CM = -.0105(7092) + .0863(80) = -67^k.$$

The above values of 83^k and 67^k for maximum tension and compression in U_3L_4 may be checked by use of formula $S = qA_z$ (2), the values of q being taken from Table 16.

Tension U_3L_4 by Equivalent Uniform Load.

The area of the tension part of the influence line equals

$$A_z = 27.2$$

The influence line *ohkm* is not triangular, but a triangular influence line with intervals $l_1 = 10$ ft. and $l_2 = 45$ ft. approximates its shape closely enough for the selection of an equivalent uniform load. For $l_1 = 10'$ and $l_2 = 45'$, Table 16 gives 3.080^k as the equivalent uniform load.

Therefore,

$$S = qA_s = (3.080)\ (27.2) = 84^k.$$

This value checks very closely that obtained by the exact method.

Compression U_3L_4 by Equivalent Uniform Load.

Choose from Table 16 the equivalent uniform load for $l_1 = 10$ ft. and $l_2 = 65$ ft. From the influence line $A_s = 23.7$.

Therefore,

$$S = qA_s = (2.870)\ (23.7) = 68^k.$$

This checks closely the value obtained by the exact method.

Calculation of other members of this arch and of some more complicated framed structures shows a close agreement between the two preceding methods. The latter method is the simpler when a table of equivalent uniform loads has been made, especially in the case of the more complex influence lines for members of swing bridges, two-hinged arches, arch ribs, etc. The method of calculating a table of equivalent uniform loads will be explained in the following article.

ARTICLE IX.

EQUIVALENT UNIFORM LOADS.

An equivalent uniform load is one which gives the same stress as does a loading which is not uniform. For any given standard loading, the equivalent uniform load is different for stresses whose influence lines differ. Since the forms of influence lines are innumerable, a table of exact equivalent uniform loads for all stresses is impracticable. A table of equivalent uniform loads, however, for stresses whose influence lines are *triangular* may be used with little error in selecting equivalent uniform loads for stresses whose influence lines are *not triangular*. It is, therefore, sufficient for practical purposes to make tables of equivalent uniform loads for a series of *triangular* influence lines. It may be shown that the equivalent uniform load for any triangular influence line is dependent entirely upon the intervals l_1 and l_2, and is independent of the ordinate h at the apex of the influence line. Consider the triangular influence line in Fig. 1b to be for any stress S. Let the ordinate below C be any value h. If q equals the equivalent uniform load covering l_1 and l_2,

$$S = qA_z, \text{ or } q = \frac{S}{A_z} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (A)$$

The area of this influence line is

$$A_z = \frac{h}{2}(l_1 + l_2) = \frac{h}{2} L \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (B)$$

Furthermore, if the concentrated live loads have been placed so as to give the maximum pier reaction between two spans l_1 and l_2, this same position of loading will give maximum S, if the influence line for S is a triangle with the

same intervals l_1 and l_2. Since the influence ordinates for S are related to the influence ordinates for R as h is to unity,

$$\frac{S}{R} = \frac{h}{1.00}$$

Or

$$S = hR \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (C)$$

Substituting the values of A_1 and S from equations (B) and (C) in equation (A),

$$q = hR + \frac{h}{2}L = \frac{2R}{L} \quad . \quad . \quad . \quad . \quad . \quad (D)$$

It appears, therefore, that q is independent of h.
From formula (16) of Art. 5,

$$R = \frac{L}{l_1 l_2} M \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

Substituting for R in equation (D),

$$q = \frac{2R}{L} = \frac{2M}{l_1 l_2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

The term M is the bending moment in the span $L = l_1 + l_2$ at the point where the intervals are l_1 and l_2.

Tables (10) to (18) inclusive have been calculated for the positions of the live load given by Table 3. The values of M were first found, then the values of R, and finally the values of the equivalent uniform loads. The three formulas that were used in succession are

$$M = \frac{l_1}{L} M_3 + \frac{l_3}{L} M_1 - M_2 \quad . \quad . \quad . \quad . \quad . \quad (10)$$

$$R = \frac{L}{l_1 l_2} M \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

$$q = \frac{2M}{l_1 l_2} = \frac{2R}{L} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

An example of the use of equivalent uniform loads has already been given in Art. 8. The general formula $S = qA_z$ may be used in any case. For the special cases of bending moment in a beam and pier reaction between two simple spans, formula (31) gives

$$M = q\left(\frac{l_1 l_2}{2}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

$$R = q\left(\frac{L}{2}\right) = q\left(\frac{l_1 + l_2}{2}\right). \quad . \quad . \quad . \quad . \quad . \quad . \quad (33)$$

The quantities in the parentheses are the areas of the influence lines for M and R respectively.

ARTICLE X.

METHOD OF CALCULATING TABLE OF LOAD SUMS FOR ANY STANDARD LOADING. ILLUSTRATIVE EXAMPLE.

THE definitions of *moment sum* and *load sum* are give. at the beginning of Art. 2. It is at once evident that a table of *load sums* may be computed by adding the successive loads. It may be shown that the table of moment sums may also be calculated by the process of addition.

From formula (5a) of Art. 2,

$$C_a W_a = C_a \frac{dM_a}{dx}$$

Or

$$dM_a = W_a \cdot dx.$$

Expressed in words, the increase in the *moment sum* for an increase dx in the distance of the centre of moments from wheel 1 equals the *load sum* times dx. If the load sum is constant for an interval $dx = 1$ foot, as between concentrated loads, the increase of the moment sum for $dx = 1$ foot equals the corresponding load sum. If the load sum is not constant, but *uniformly* increasing, as when the centre of moments lies within the uniform load, the increase of the moment sum for $dx = 1$ foot equals the *average* value of the load sum for this one foot interval. The application of the foregoing principles is made clear by the following example.

Example.—Give explicit directions for the calculation of a table of load sums and moment sums at intervals of 1 foot from 0′ to 400′ for Cooper's $E40$ loading.

Solution.—Calculate the table of load sums by adding

the loads one by one, taking a sub-total for each addition. Thus, the following numbers are added:

1—10
4—20's
4—13's
1—10
4—20's
4—13's
391— 2's

If the final total checks $284 + 391 \times 2 = 866$, the table of load sums is correct.

Assume now that the table of load sums for $E40$ has been completed. The table of moment sums may now be found as directed below. The following numbers are to be added one by one, taking a sub-total for each addition:

8—10's
5—30's
5—50's
5—70's
9—90's
5—103's
6—116's
5—129's
8—142's
8—152's
5—172's
5—192's
5—212's
9—232's
5—245's
6—258's
5—271's
5—284's
1—285
1—287
1—289

and all odd numbers up to 865.

If the final total checks up 183,689, which is figured independently, the table of moment sums is correct.

The preceding additions may be made most satisfactorily on a recording adding machine. Table 2 was calculated in this way.

It will be noted that the table of load sums serves as a table of differences for the table of moment sums.

ARTICLE XI.

SUMMARY OF FORMULAS.

Art. 1.

$$Z = \Sigma wz \quad \ldots \quad (1)$$

$$Z = qA_z \quad \ldots \quad (2)$$

$$Z = w\Sigma z \quad \ldots \quad (3)$$

$$Z = z\Sigma w = zW \quad \ldots \quad (4)$$

Art. 2.

$$Z = \Sigma w_n z_n = C_a \Sigma w_n x_n = C_a M_a \quad \ldots \quad (5)$$

$$\frac{dZ}{dx} = C_a W_a = \frac{d\,(C_a M_a)}{dx} = \frac{C_a dM_a}{dx} \quad \ldots \quad (5a)$$

Art. 3.

$$\Sigma C = 0 \quad \ldots \quad (6)$$

$$S = \Sigma CM \quad \ldots \quad (7)$$

$$\frac{dS}{dx} = \Sigma CW \quad \ldots \quad (8)$$

Art. 4. *Girder Bridge without Panels.*

End reactions.

$$R_1 = \frac{M_2 - M_1}{L} - W_1 \quad \ldots \quad (9)$$

$$R_2 = W_2 - \frac{M_2 - M_1}{L} \quad \ldots \quad (9a)$$

Bending moment for unequal segments l_1 and l_2.

$$M = \frac{l_1}{L} M_2 + \frac{l_2}{L} M_1 - M_x \quad \ldots \quad (10)$$

$$\frac{dM}{dx} = \frac{l_1}{L} W_2 + \frac{l_2}{L} W_1 - W_x \quad \ldots \quad (11)$$

Bending moment at centre. $l_1 = l_2 = \frac{L}{2}$

$$M = \frac{M_3 + M_1}{2} - M_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10a)$$

$$\frac{dM}{dx} = \frac{W_3 + W_1}{2} - W_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11a)$$

Shear at any section.

$$S = \frac{M_3 - M_1}{L} - W_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

Location of centre of gravity of loading on span.

$$\bar{x} = \frac{M_3 - M_1 - LW_1}{W_3 - W_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

When $M_1 = 0$,

$$\bar{x} = \frac{M_3}{W_3} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13a)$$

Art. 5. Pier Reaction.

For unequal spans l_1 and l_2.

$$R = \frac{M_3}{l_2} + \frac{M_1}{l_1} - \frac{L}{l_1 l_2} M_2 = \frac{L}{l_1 l_2}\left(\frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2\right) \quad (14)$$

$$\frac{dR}{dx} = \frac{W_3}{l_2} + \frac{W_1}{l_1} - \frac{L}{l_1 l_2} W_2 = \frac{L}{l_1 l_2}\left(\frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2\right) \quad (15)$$

For equal spans l_1 and l_2 equal to l.

$$R = \frac{M_3 + M_1 - 2M_2}{l} \quad . \quad . \quad . \quad . \quad . \quad . \quad (14a)$$

$$\frac{dR}{dx} = \frac{W_3 + W_1 - 2W_2}{l} \quad . \quad . \quad . \quad . \quad . \quad . \quad (15a)$$

Relation between R and M,

$$R = \frac{L}{l_1 l_2} M \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

Art. 6. Girder Bridge with Panels.

Shear in end panel; general case.

$$S_a = \frac{1}{L} M_3 + \frac{l_2}{l_1 L} M_1 - \frac{1}{l_1} M_2 = \frac{1}{l_1}\left(\frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2\right) \quad (17)$$

$$\frac{dS_a}{dx} = \frac{1}{L}W_3 + \frac{l_3}{l_1 L}W_1 - \frac{1}{l_1}W_2 = \frac{1}{l_1}\left(\frac{l_1}{L}W_3 + \frac{l_3}{L}W_1 - W_2\right) \quad (18)$$

Shear in intermediate panel; general case.

$$S_b = \frac{M_4}{L} - \frac{M_3}{p} + \frac{M_2}{p} - \frac{M_1}{L} \quad . \quad . \quad . \quad . \quad (19)$$

$$\frac{dS_b}{dx} = \frac{W_4}{L} - \frac{W_3}{p} + \frac{W_2}{p} - \frac{W_1}{L} \quad . \quad . \quad . \quad (20)$$

Shear in intermediate panel; usual case.

$$S = \frac{M_4}{L} - \frac{M_3}{p} = \frac{1}{p}\left(\frac{p}{L}M_4 - M_3\right) \quad (19a)$$

$$\frac{dS_b}{dx} = \frac{W_4}{L} - \frac{W_3}{p} = \frac{1}{p}\left(\frac{p}{L}W_4 - W_3\right) \quad (20a)$$

Art. 7. Through Pratt Truss with Inclined Chord.

Stress in hanger. Use formulas (14a) and (15a).

Stress in any horizontal chord member; usual case.

$$S_3 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \quad . \quad . \quad . \quad (21)$$

Compression in any inclined top chord member or end post; usual case.

$$S_6 = \left(\frac{i}{p}\right)S_5 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

Compression in vertical post; usual case.

$$S_4 = \left(\frac{a}{bL}\right)M_4 - \left(\frac{1}{p}\right)M_3 \quad . \quad . \quad . \quad . \quad . \quad . \quad (23)$$

Stresses in inclined web members including counters; usual case.

$$S_1, S_2, S_3 = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_3 \quad . \quad (24)$$

Stress in inclined counter; special case of loading advanced beyond panel.

$$S_2 = \left(\frac{ta}{cbL}\right)M_4 - \frac{t}{bp}\left(M_3 - \frac{b}{c}M_2\right) = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_c \quad (25)$$

Counter-tension in vertical post; usual case.

$$T = \left(\frac{d_2 - d_1}{bp}\right)\left(\frac{m}{n}M_4 - M_3\right) = K \cdot M_o \quad . \; . \; (26)$$

Formulas (21), (23), and (24) are of the general form

$$S = GM_4 - HM_3 \quad . \; . \; . \; . \; . \; . \; . \; (27)$$

where the coefficients G and H may be tabulated thus:

Type of member	G	H
Horizontal chord	$\frac{m}{nv}$	$\frac{1}{v}$
Vertical post	$\frac{a}{bL}$	$\frac{1}{p}$
Inclined web member	$\frac{ta}{cbL}$	$\frac{t}{bp}$

The rate of variation of S in formula (27) is

$$\frac{dS}{dx} = GW_4 - HW_3 = H\left(\frac{G}{H}W_4 - W_3\right) \quad . \; . \; (28)$$

When S in formulas (21), (23), or (24) is a maximum

$$\left(\frac{G}{H}W_4 - W_3\right) \text{ passes through zero.}$$

Through Pratt Truss—Parallel Chords.

Stress in hanger,—use formulas (14a) and (15a)

$$\text{Stress in horizontal chord} = S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \; (21)$$

$$\text{“ “ vertical post} = S_4 = \left(\frac{1}{L}\right)M_4 - \left(\frac{1}{p}\right)M_3 \quad . \; . \; (29)$$

$$\text{“ “ inclined web} = S_1 = \left(\frac{t}{cL}\right)M_4 - \left(\frac{t}{cp}\right)M_3 = \frac{t}{c}S_4 \quad (30)$$

Stress in end post $= S_4 = \frac{\text{[illegible]}}{p} S_3$ . . . . (22)

Formulas (21), (29), and (30) are of the general form

$$S = G \cdot M_4 - H \cdot M_3 \qquad (27)$$

and their rate of variation is

$$\frac{dS}{dx} = H\left(\frac{G}{H} W_4 - W_3\right) \qquad (28)$$

G and H are the coefficients of M_4 and M_3 in equations (21), (29), and (30), respectively.

When S in formulas (21), (29), or (30) is a maximum, $\left(\frac{G}{H} W_4 - W_3\right)$ passes through zero.

Art. 9. Equivalent Uniform Loads.

$$q = \frac{2M}{l_1 l_2} = \frac{2R}{L} \qquad (31)$$

$$M = q\left(\frac{l_1 l_2}{2}\right) \qquad (32)$$

$$R = q\left(\frac{L}{2}\right) = q\left(\frac{l_1 + l_2}{2}\right) \qquad (33)$$

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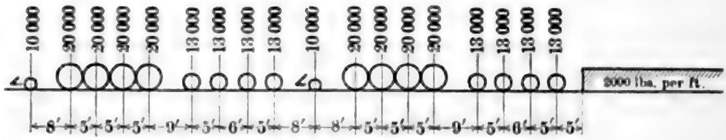
TABLE 1

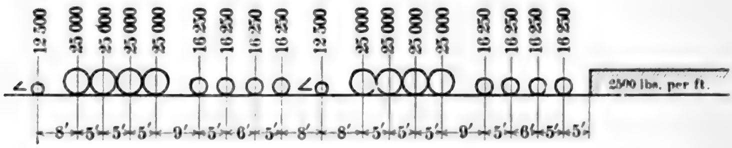
STANDARD LOADINGS

Loads given are for one rail.

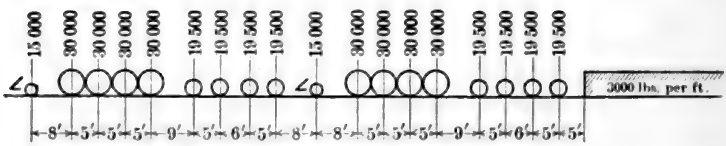
COOPER'S E 40:



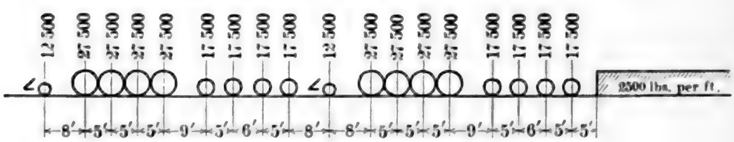
COOPER'S E 50:



COOPER'S E 60:



COMMON STANDARD-1904-PACIFIC SYSTEM



D. L. & W. R. R.:

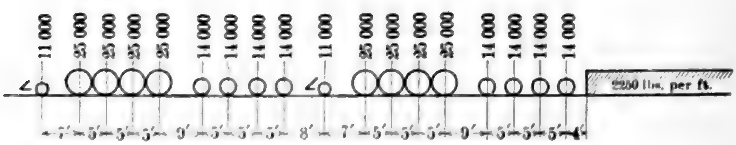


TABLE 2

LOAD SUMS AND MOMENT SUMS FOR COOPER'S AND OTHER STANDARD LOADINGS

NOTE.—Load Sums and Moment Sums are given per rail in thousands of pounds and foot-pounds respectively.

COOPER'S *E*40. 0′–50′ COOPER'S *E*40. 50′–100′

Length	Wheel	Load	Load Sums	Moment Sums	Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	10	10	0	50	......	..	...	3780
1	....	..	...	10	51	......	..	...	3922
2	....	..	...	20	52	......	..	...	4064
3	....	..	...	30	53	......	..	...	4206
4	....	..	...	40	54	......	..	...	4348
5	....	..	...	50	55	......	..	...	4490
6	....	..	...	60	56	w. 10	10	152	4632
7	....	..	...	70	57	......	..	...	4784
8	w. 2	20	30	80	58	......	..	...	4936
9	....	..	...	110	59	......	..	...	5088
10	....	..	...	140	60	......	..	...	5240
11	....	..	...	170	61	......	..	...	5392
12	....	..	...	200	62	......	..	...	5544
13	w. 3	20	50	230	63	......	..	...	5696
14	....	..	...	280	64	w. 11	20	172	5848
15	....	..	...	330	65	......	..	...	6020
16	....	..	...	380	66	......	..	...	6192
17	....	..	...	430	67	......	..	...	6364
18	w. 4	20	70	480	68	......	..	...	6536
19	....	..	...	550	69	w. 12	20	192	6708
20	....	..	...	620	70	......	..	...	6900
21	....	..	...	690	71	......	..	...	7092
22	....	..	...	760	72	......	..	...	7284
23	w. 5	20	90	830	73	......	..	...	7476
24	....	..	...	920	74	w. 13	20	212	7668
25	....	..	...	1010	75	......	..	...	7880
26	....	..	...	1100	76	......	..	...	8092
27	....	..	...	1190	77	......	..	...	8304
28	....	..	...	1280	78	......	..	...	8516
29	....	..	...	1370	79	w. 14	20	232	8728
30	....	..	...	1460	80	......	..	...	8960
31	....	..	...	1550	81	......	..	...	9192
32	w. 6	13	103	1640	82	......	..	...	9424
33	....	..	...	1743	83	......	..	...	9656
34	....	..	...	1846	84	......	..	...	9888
35	....	..	...	1949	85	......	..	...	10120
36	....	..	...	2052	86	......	..	...	10352
37	w. 7	13	116	2155	87	......	..	...	10584
38	....	..	...	2271	88	w. 15	13	245	10816
39	....	..	...	2387	89	......	..	...	11061
40	....	..	...	2503	90	......	..	...	11306
41	....	..	...	2619	91	......	..	...	11551
42	....	..	...	2735	92	......	..	...	11796
43	w. 8	13	129	2851	93	w. 16	13	258	12041
44	....	..	...	2980	94	......	..	...	12299
45	....	..	...	3109	95	......	..	...	12557
46	....	..	...	3238	96	......	..	...	12815
47	....	..	...	3367	97	......	..	...	13073
48	w. 9	13	142	3496	98	......	..	...	13331
49	....	..	...	3638	99	w. 17	13	271	13589
50	....	..	...	3780	100	......	..	...	13860

Cooper's *E*40. 100′–150′

Length	Wheel	Load	Load Sums	Moment Sums
100	......	......		13860
101	......	......		14131
102	......	......		14402
103	......	......		14673
104	w. 18	13	284	14944
105	......	......		15228
106	......	......		15512
107	......	......		15796
108	......	......		16080
109	......	......	284	16364
110	......	Uniform Load = 2,000 pounds per foot	286	16649
111	......		288	16936
112	......		290	17225
113	......		292	17516
114	......		294	17809
115	......		296	18104
116	......		298	18401
117	......		300	18700
118	......		302	19001
119	......		304	19304
120	......		306	19609
121	......		308	19916
122	......		310	20225
123	......		312	20536
124	......		314	20849
125	......		316	21164
126	......		318	21481
127	......		320	21800
128	......		322	22121
129	......		324	22444
130	......		326	22769
131	......		328	23096
132	......		330	23425
133	......		332	23756
134	......		334	24089
135	......		336	24424
136	......		338	24761
137	......		340	25100
138	......		342	25441
139	......		344	25784
140	......		346	26129
141	......		348	26476
142	......		350	26825
143	......		352	27176
144	......		354	27529
145	......		356	27884
146	......		358	28241
147	......		360	28600
148	......		362	28961
149	......		364	29324
150	......		366	29689

Cooper's *E*40. 150′–200′

Length	Load	Load Sums	Moment Sums
150	Uniform Load = 2,000 pounds per foot	366	29689
151		368	30056
152		370	30425
153		372	30796
154		374	31169
155		376	31544
156		378	31921
157		380	32300
158		382	32681
159		384	33064
160		386	33449
161		388	33836
162		390	34225
163		392	34616
164		394	35009
165		396	35404
166		398	35801
167		400	36200
168		402	36601
169		404	37004
170		406	37409
171		408	37816
172		410	38225
173		412	38636
174		414	39049
175		416	39464
176		418	39881
177		420	40300
178		422	40721
179		424	41144
180		426	41569
181		428	41996
182		430	42425
183		432	42856
184		434	43289
185		436	43724
186		438	44161
187		440	44600
188		442	45041
189		444	45484
190		446	45929
191		448	46376
192		450	46825
193		452	47276
194		454	47729
195		456	48184
196		458	48641
197		460	49100
198		462	49561
199		464	50024
200		466	50489

Cooper's *E*40. 200′–250′				Cooper's *E*40. 250′–300′			
Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
200	Uniform Load = 2,000 pounds per foot	466	50489	250	Uniform Load = 2,000 pounds per foot	566	76289
201		468	50956	251		568	76856
202		470	51425	252		570	77425
203		472	51896	253		572	77996
204		474	52369	254		574	78569
205		476	52844	255		576	79144
206		478	53321	256		578	79721
207		480	53800	257		580	80300
208		482	54281	258		582	80881
209		484	54764	259		584	81464
210		486	55249	260		586	82049
211		488	55736	261		588	82636
212		490	56225	262		590	83225
213		492	56716	263		592	83816
214		494	57209	264		594	84409
215		496	57704	265		596	85004
216		498	58201	266		598	85601
217		500	58700	267		600	86200
218		502	59201	268		602	86801
219		504	59704	269		604	87404
220		506	60209	270		606	88009
221		508	60716	271		608	88616
222		510	61225	272		610	89225
223		512	61736	273		612	89836
224		514	62249	274		614	90449
225		516	62764	275		616	91064
226		518	63281	276		618	91681
227		520	63800	277		620	92300
228		522	64321	278		622	92921
229		524	64844	279		624	93544
230		526	65369	280		626	94169
231		528	65896	281		628	94796
232		530	66425	282		630	95425
233		532	66956	283		632	96056
234		534	67489	284		634	96689
235		536	68024	285		636	97324
236		538	68561	286		638	97961
237		540	69100	287		640	98600
238		542	69641	288		642	99241
239		544	70184	289		644	99884
240		546	70729	290		646	100529
241		548	71276	291		648	101176
242		550	71825	292		650	101825
243		552	72376	293		652	102476
244		554	72929	294		654	103129
245		556	73484	295		656	103784
246		558	74041	296		658	104441
247		560	74600	297		660	105100
248		562	75161	298		662	105761
249		564	75724	299		664	106424
250		566	76289	300		666	107089

Cooper's E40. 300′–350′				Cooper's E40. 350′–400′			
Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
300	Uniform Load = 2,000 pounds per foot	666	107089	350	Uniform Load = 2,000 pounds per foot	766	142889
301		668	107756	351		768	143656
302		670	108425	352		770	144425
303		672	109096	353		772	145196
304		674	109769	354		774	145969
305		676	110444	355		776	146744
306		678	111121	356		778	147521
307		680	111800	357		780	148300
308		682	112481	358		782	149081
309		684	113164	359		784	149864
310		686	113849	360		786	150649
311		688	114536	361		788	151436
312		690	115225	362		790	152225
313		692	115916	363		792	153016
314		694	116609	364		794	153809
315		696	117304	365		796	154604
316		698	118001	366		798	155401
317		700	118700	367		800	156200
318		702	119401	368		802	157001
319		704	120104	369		804	157804
320		706	120809	370		806	158609
321		708	121516	371		808	159416
322		710	122225	372		810	160225
323		712	122936	373		812	161036
324		714	123649	374		814	161849
325		716	124364	375		816	162664
326		718	125081	376		818	163481
327		720	125800	377		820	164300
328		722	126521	378		822	165121
329		724	127244	379		824	165944
330		726	127969	380		826	166769
331		728	128696	381		828	167596
332		730	129425	382		830	168425
333		732	130156	383		832	169256
334		734	130889	384		834	170089
335		736	131624	385		836	170924
336		738	132361	386		838	171761
337		740	133100	387		840	172600
338		742	133841	388		842	173441
339		744	134584	389		844	174284
340		746	135329	390		846	175129
341		748	136076	391		848	175976
342		750	136825	392		850	176825
343		752	137576	393		852	177676
344		754	138329	394		854	178529
345		756	139084	395		856	179384
346		758	139841	396		858	180241
347		760	140600	397		860	181100
348		762	141361	398		862	181961
349		764	142124	399		864	182824
350		766	142889	400		866	183689

Cooper's $E50$. 0′–50′

Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	12.50	12.50	00.00
1	....	....	....	12.50
2	....	....	....	25.00
3	....	....	....	37.50
4	....	....	....	50.00
5	....	....	....	62.50
6	....	....	....	75.00
7	....	....	....	87.50
8	w. 2	25.00	37.50	100.00
9	....	....	....	137.50
10	....	....	....	175.00
11	....	....	....	212.50
12	....	....	....	250.00
13	w. 3	25.00	62.50	287.50
14	....	....	....	350.00
15	....	....	....	412.50
16	....	....	....	475.00
17	....	....	....	537.50
18	w. 4	25.00	87.50	600.00
19	....	....	....	687.50
20	....	....	....	775.00
21	....	....	....	862.50
22	....	....	....	950.00
23	w. 5	25.00	112.50	1037.50
24	....	....	....	1150.00
25	....	....	....	1262.50
26	....	....	....	1375.00
27	....	....	....	1487.50
28	....	....	....	1600.00
29	....	....	....	1712.50
30	....	....	....	1825.00
31	....	....	....	1937.50
32	w. 6	16.25	128.75	2050.00
33	....	....	....	2178.75
34	....	....	....	2307.50
35	....	....	....	2436.25
36	....	....	....	2565.00
37	w. 7	16.25	145.00	2693.75
38	....	....	....	2838.75
39	....	....	....	2983.75
40	....	....	....	3128.75
41	....	....	....	3273.75
42	....	....	....	3418.75
43	w. 8	16.25	161.25	3563.75
44	....	....	....	3725.00
45	....	....	....	3886.25
46	....	....	....	4047.50
47	....	....	....	4208.75
48	w. 9	16.25	177.50	4370.00
49	....	....	....	4547.50
50	....	....	....	4725.00

Cooper's $E50$. 50′–100′

Length	Wheel	Load	Load Sums	Moment Sums
50	....	....	....	4725.00
51	....	....	....	4902.50
52	....	....	....	5080.00
53	....	....	....	5257.50
54	....	....	....	5435.00
55	....	....	....	5612.50
56	w. 10	12.50	190.00	5790.00
57	....	....	....	5980.00
58	....	....	....	6170.00
59	....	....	....	6360.00
60	....	....	....	6550.00
61	....	....	....	6740.00
62	....	....	....	6930.00
63	....	....	....	7120.00
64	w. 11	25.00	215.00	7310.00
65	....	....	....	7525.00
66	....	....	....	7740.00
67	....	....	....	7955.00
68	....	....	....	8170.00
69	w. 12	25.00	240.00	8385.00
70	....	....	....	8625.00
71	....	....	....	8865.00
72	....	....	....	9105.00
73	....	....	....	9345.00
74	w. 13	25.00	265.00	9585.00
75	....	....	....	9850.00
76	....	....	....	10115.00
77	....	....	....	10380.00
78	....	....	....	10645.00
79	w. 14	25.00	290.00	10910.00
80	....	....	....	11200.00
81	....	....	....	11490.00
82	....	....	....	11780.00
83	....	....	....	12070.00
84	....	....	....	12360.00
85	....	....	....	12650.00
86	....	....	....	12940.00
87	....	....	....	13230.00
88	w. 15	16.25	306.25	13520.00
89	....	....	....	13826.25
90	....	....	....	14132.50
91	....	....	....	14438.75
92	....	....	....	14745.00
93	w. 16	16.25	322.50	15051.25
94	....	....	....	15373.75
95	....	....	....	15696.25
96	....	....	....	16018.75
97	....	....	....	16341.25
98	....	....	....	16663.75
99	w. 17	16.25	338.75	16986.25
100	....	....	....	17325.00

Cooper's E50. 100′–150′

Length	Wheel	Load	Load Sums	Moment Sums
100	......	.....	......	17325.00
101	......	.....	......	17663.75
102	......	.....	......	18002.50
103	......	.....	......	18341.25
104	w. 18	16.25	355.00	18680.00
105	......	.....	......	19035.00
106	......	.....	......	19390.00
107	......	.....	......	19745.00
108	......	.....	......	20100.00
109	......	.....	355.00	20455.00
110	......	Uniform Load = 2,500 pounds per foot	357.50	20811.25
111	......		360.00	21170.00
112	......		362.50	21531.25
113	......		365.00	21895.00
114	......		367.50	22261.25
115	......		370.00	22630.00
116	......		372.50	23001.25
117	......		375.00	23375.00
118	......		377.50	23751.25
119	......		380.00	24130.00
120	......		382.50	24511.25
121	......		385.00	24895.00
122	......		387.50	25281.25
123	......		390.00	25670.00
124	......		392.50	26061.25
125	......		395.00	26455.00
126	......		397.50	26851.25
127	......		400.00	27250.00
128	......		402.50	27651.25
129	......		405.00	28055.00
130	......		407.50	28461.25
131	......		410.00	28870.00
132	......		412.50	29281.25
133	......		415.00	29695.00
134	......		417.50	30111.25
135	......		420.00	30530.00
136	......		422.50	30951.25
137	......		425.00	31375.00
138	......		427.50	31801.25
139	......		430.00	32230.00
140	......		432.50	32661.25
141	......		435.00	33095.00
142	......		437.50	33531.25
143	......		440.00	33970.00
144	......		442.50	34411.00
145	......		445.00	34855.00
146	......		447.50	35301.25
147	......		450.00	35750.00
148	......		452.50	36201.25
149	......		455.00	36655.00
150	......		457.50	37111.25

Cooper's E50. 150′–200′

Length	Load	Load Sums	Moment Sums
150	Uniform Load = 2,500 pounds per foot	457.50	37111.25
151		460.00	37570.00
152		462.50	38031.25
153		465.00	38495.00
154		467.50	38961.25
155		470.00	39430.00
156		472.50	39901.25
157		475.00	40375.00
158		477.50	40851.25
159		480.00	41330.00
160		482.50	41811.25
161		485.00	42295.00
162		487.50	42781.25
163		490.00	43270.00
164		492.50	43761.25
165		495.00	44255.00
166		497.50	44751.25
167		500.00	45250.00
168		502.50	45751.25
169		505.00	46255.00
170		507.50	46761.25
171		510.00	47270.00
172		512.50	47781.25
173		515.00	48295.00
174		517.50	48811.25
175		520.00	49330.00
176		522.50	49851.25
177		525.00	50375.00
178		527.50	50901.25
179		530.00	51430.00
180		532.50	51961.25
181		535.00	52495.00
182		537.50	53031.25
183		540.00	53570.00
184		542.50	54111.25
185		545.00	54655.00
186		547.50	55201.25
187		550.00	55750.00
188		552.50	56301.25
189		555.00	56855.00
190		557.50	57411.25
191		560.00	57970.00
192		562.50	58531.25
193		565.00	59095.00
194		567.50	59661.25
195		570.00	60230.00
196		572.50	60801.25
197		575.00	61375.00
198		577.50	61951.25
199		580.00	62530.00
200		582.50	63111.25

Cooper's $E50$. 200′–250′				Cooper's $E50$. 250′–300			
Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
200	Uniform Load = 2,500 pounds per foot	582.50	63111.25	250	Uniform Load = 2,500 pounds per foot	707.50	95361.25
201		585.00	63695.00	251		710.00	96070.00
202		587.50	64281.25	252		712.50	96781.25
203		590.00	64870.00	253		715.00	97495.00
204		592.50	65461.25	254		717.50	98211.25
205		595.00	66055.00	255		720.00	98930.00
206		597.50	66651.25	256		722.50	99651.25
207		600.00	67250.00	257		725.00	100375.00
208		602.50	67851.25	258		727.50	101101.25
209		605.00	68455.00	259		730.00	101830.00
210		607.50	69061.25	260		732.50	102561.25
211		610.00	69670.00	261		735.00	103295.00
212		612.50	70281.25	262		737.50	104031.25
213		615.00	70895.00	263		740.00	104770.00
214		617.50	71511.25	264		742.50	105511.25
215		620.00	72130.00	265		745.00	106255.00
216		622.50	72751.25	266		747.50	107001.25
217		625.00	73375.00	267		750.00	107750.00
218		627.50	74001.25	268		752.50	108501.25
219		630.00	74630.00	269		755.00	109255.00
220		632.50	75261.25	270		757.50	110011.25
221		635.00	75895.00	271		760.00	110770.00
222		637.50	76531.25	272		762.50	111531.25
223		640.00	77170.00	273		765.00	112295.00
224		642.50	77811.25	274		767.50	113061.25
225		645.00	78455.00	275		770.00	113830.00
226		647.50	79101.25	276		772.50	114601.25
227		650.00	79750.00	277		775.00	115375.00
228		652.50	80401.25	278		777.50	116151.25
229		655.00	81055.00	279		780.00	116930.00
230		657.50	81711.25	280		782.50	117711.25
231		660.00	82370.00	281		785.00	118495.00
232		662.50	83031.25	282		787.50	119281.25
233		665.00	83695.00	283		790.00	120070.00
234		667.50	84361.25	284		792.50	120861.25
235		670.00	85030.00	285		795.00	121655.00
236		672.50	85701.25	286		797.50	122451.25
237		675.00	86375.00	287		800.00	123250.00
238		677.50	87051.25	288		802.50	124051.25
239		680.00	87730.00	289		805.00	124855.00
240		682.50	88411.25	290		807.50	125661.25
241		685.00	89095.00	291		810.00	126470.00
242		687.50	89781.25	292		812.50	127281.25
243		690.00	90470.00	293		815.00	128095.00
244		692.50	91161.25	294		817.50	128911.25
245		695.00	91855.00	295		820.00	129730.00
246		697.50	92551.25	296		822.50	130551.25
247		700.00	93250.00	297		825.00	131375.00
248		702.50	93951.25	298		827.50	132201.25
249		705.00	94655.00	299		830.00	133030.00
250		707.50	95361.25	300		832.50	133861.25

Cooper's *E*50. 300′–350′

Length	Load	Load Sums	Moment Sums
300	Uniform Load = 2,500 pounds per foot	832.50	133861.25
301		835.00	134695.00
302		837.50	135531.25
303		840.00	136370.00
304		842.50	137211.25
305		845.00	138055.00
306		847.50	138901.25
307		850.00	139750.00
308		852.50	140601.25
309		855.00	141455.00
310		857.50	142311.25
311		860.00	143170.00
312		862.50	144031.25
313		865.00	144895.00
314		867.50	145761.25
315		870.00	146630.00
316		872.50	147501.25
317		875.00	148375.00
318		877.50	149251.25
319		880.00	150130.00
320		882.50	151011.25
321		885.00	151895.00
322		887.50	152781.25
323		890.00	153670.00
324		892.50	154561.25
325		895.00	155455.00
326		897.50	156351.25
327		900.00	157250.00
328		902.50	158151.25
329		905.00	159055.00
330		907.50	159961.25
331		910.00	160870.00
332		912.50	161781.25
333		915.00	162695.00
334		917.50	163611.25
335		920.00	164530.00
336		922.50	165451.25
337		925.00	166375.00
338		927.50	167301.25
339		930.00	168230.00
340		932.50	169161.25
341		935.00	170095.00
342		937.50	171031.25
343		940.00	171970.00
344		942.50	172911.25
345		945.00	173855.00
346		947.50	174801.25
347		950.00	175750.00
348		952.50	176701.25
349		955.00	177655.00
350		957.50	178611.25

Cooper's *E*50. 350′–400′

Length	Load	Load Sums	Moment Sums
350	Uniform Load = 2,500 pounds per foot	957.50	178611.25
351		960.00	179570.00
352		962.50	180531.25
353		965.00	181495.00
354		967.50	182461.25
355		970.00	183430.00
356		972.50	184401.25
357		975.00	185375.00
358		977.50	186351.25
359		980.00	187330.00
360		982.50	188311.25
361		985.00	189295.00
362		987.50	190281.25
363		990.00	191270.00
364		992.50	192261.25
365		995.00	193255.00
366		997.50	194251.25
367		1000.00	195250.00
368		1002.50	196251.25
369		1005.00	197255.00
370		1007.50	198261.25
371		1010.00	199270.00
372		1012.50	200281.25
373		1015.00	201295.00
374		1017.50	202311.25
375		1020.00	203330.00
376		1022.50	204351.25
377		1025.00	205375.00
378		1027.50	206401.25
379		1030.00	207430.00
380		1032.50	208461.25
381		1035.00	209495.00
382		1037.50	210531.25
383		1040.00	211570.00
384		1042.50	212611.25
385		1045.00	213655.00
386		1047.50	214701.25
387		1050.00	215750.00
388		1052.50	216801.25
389		1055.00	217855.00
390		1057.50	218911.25
391		1060.00	219970.00
392		1062.50	221031.25
393		1065.00	222095.00
394		1067.50	223161.25
395		1070.00	224230.00
396		1072.50	225301.25
397		1075.00	226375.00
398		1077.50	227451.25
399		1080.00	228530.00
400		1082.50	229611.25

Cooper's *E*60. 0′–50′					Cooper's *E*60. 50′–100′				
Length	Wheel	Load	Load Sums	Moment Sums	Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	15.0	15.0	00.00	50	....	....	....	5670.00
1	....	....	....	15.00	51	....	....	....	5883.00
2	....	....	....	30.00	52	....	....	....	6096.00
3	....	....	....	45.00	53	....	....	....	6309.00
4	....	....	....	60.00	54	....	....	....	6522.00
5	....	....	....	75.00	55	....	....	....	6735.00
6	....	....	....	90.00	56	w. 10	15.0	228.0	6948.00
7	....	....	....	105.00	57	....	....	....	7176.00
8	w. 2	30.0	45.0	120.00	58	....	....	....	7404.00
9	....	....	....	165.00	59	....	....	....	7632.00
10	....	....	....	210.00	60	....	....	....	7860.00
11	....	....	....	255.00	61	....	....	....	8088.00
12	....	....	....	300.00	62	....	....	....	8316.00
13	w. 3	30.0	75.0	345.00	63	....	....	....	8544.00
14	....	....	....	420.00	64	w. 11	30.0	258.0	8772.00
15	....	....	....	495.00	65	....	....	....	9030.00
16	....	....	....	570.00	66	....	....	....	9288.00
17	....	....	....	645.00	67	....	....	....	9546.00
18	w. 4	30.0	105.0	720.00	68	....	....	....	9804.00
19	....	....	....	825.00	69	w. 12	30.0	288.0	10062.00
20	....	....	....	930.00	70	....	....	....	10350.00
21	....	....	....	1035.00	71	....	....	....	10638.00
22	....	....	....	1140.00	72	....	....	....	10926.00
23	w. 5	30.0	135.0	1245.00	73	....	....	....	11214.00
24	....	....	....	1380.00	74	w. 13	30.0	318.0	11502.00
25	....	....	....	1515.00	75	....	....	....	11820.00
26	....	....	....	1650.00	76	....	....	....	12138.00
27	....	....	....	1785.00	77	....	....	....	12456.00
28	....	....	....	1920.00	78	....	....	....	12774.00
29	....	....	....	2055.00	79	w. 14	30.0	348.0	13092.00
30	....	....	....	2190.00	80	....	....	....	13440.00
31	....	....	....	2325.00	81	....	....	....	13788.00
32	w. 6	19.5	154.5	2460.00	82	....	....	....	14136.00
33	....	....	....	2614.50	83	....	....	....	14484.00
34	....	....	....	2769.00	84	....	....	....	14832.00
35	....	....	....	2923.50	85	....	....	....	15180.00
36	....	....	....	3078.00	86	....	....	....	15528.00
37	w. 7	19.5	174.0	3232.50	87	....	....	....	15876.00
38	....	....	....	3406.50	88	w. 15	19.5	367.5	16224.00
39	....	....	....	3580.50	89	....	....	....	16591.00
40	....	....	....	3754.50	90	....	....	....	16959.00
41	....	....	....	3928.50	91	....	....	....	17326.50
42	....	....	....	4102.50	92	....	....	....	17694.00
43	w. 8	19.5	193.5	4276.50	93	w. 16	19.5	387.0	18061.50
44	....	....	....	4470.00	94	....	....	....	18448.00
45	....	....	....	4663.50	95	....	....	....	18835.50
46	....	....	....	4857.00	96	....	....	....	19222.50
47	....	....	....	5050.50	97	....	....	....	19609.50
48	w. 9	19.5	213.0	5244.00	98	....	....	....	19996.50
49	....	....	....	5457.00	99	w. 17	19.5	406.5	20383.50
50	....	....	....	5670.00	100	....	....	....	20790.00

Cooper's *E*60. 100'-150'

Length	Wheel	Load	Load Sums	Moment Sums
100	......	....	.....	20790.00
101	......	....	.....	21196.50
102	......	....	.....	21603.00
103	......	....	.....	22009.50
104	w. 18	19.5	426.0	22416.00
105	......	....	.....	22842.00
106	......	....	.....	23268.00
107	......	....	.....	23694.00
108	......	....	.....	24120.00
109			426.0	24546.00
110	Uniform Load = 3,000 pounds per foot		429.0	24973.50
111			432.0	25404.00
112			435.0	25837.50
113			438.0	26274.00
114			441.0	26713.50
115			444.0	27156.00
116			447.0	27601.50
117			450.0	28050.00
118			453.0	28501.50
119			456.0	28956.00
120			459.0	29413.50
121			462.0	29874.00
122			465.0	30337.50
123			468.0	30804.00
124			471.0	31273.50
125			474.0	31746.00
126			477.0	32221.50
127			480.0	32700.00
128			483.0	33181.50
129			486.0	33666.00
130			489.0	34153.50
131			492.0	34644.00
132			495.0	35137.50
133			498.0	35634.00
134			501.0	36133.50
135			504.0	36636.00
136			507.0	37141.50
137			510.0	37650.00
138			513.0	38161.50
139			516.0	38676.00
140			519.0	39193.50
141			522.0	39714.00
142			525.0	40237.50
143			528.0	40764.00
144			531.0	41293.50
145			534.0	41826.00
146			537.0	42361.50
147			540.0	42900.00
148			543.0	43441.50
149			546.0	43986.00
150			549.0	44533.50

Cooper's *E*60 150'-200'

Length	Load	Load Sums	Moment Sums
150	Uniform Load = 3,000 pounds per foot	549.0	44533.50
151		552.0	45084.00
152		555.0	45637.50
153		558.0	46194.00
154		561.0	46753.50
155		564.0	47316.00
156		567.0	47881.50
157		570.0	48450.00
158		573.0	49021.50
159		576.0	49596.00
160		579.0	50173.50
161		582.0	50754.00
162		585.0	51337.50
163		588.0	51924.00
164		591.0	52513.50
165		594.0	53106.00
166		597.0	53701.50
167		600.0	54300.00
168		603.0	54901.50
169		606.0	55506.00
170		609.0	56113.50
171		612.0	56724.00
172		615.0	57337.50
173		618.0	57954.00
174		621.0	58573.50
175		624.0	59196.00
176		627.0	59821.50
177		630.0	60450.00
178		633.0	61081.50
179		636.0	61716.00
180		639.0	62353.50
181		642.0	62994.00
182		645.0	63637.50
183		648.0	64284.00
184		651.0	64933.50
185		654.0	65586.00
186		657.0	66241.50
187		660.0	66900.00
188		663.0	67561.50
189		666.0	68226.00
190		669.0	68893.50
191		672.0	69564.00
192		675.0	70237.50
193		678.0	70914.00
194		681.0	71593.50
195		684.0	72276.00
196		687.0	72961.50
197		690.0	73650.00
198		693.0	74341.50
199		696.0	75036.00
200		699.0	75733.50

Cooper's *E*60. 200′–250′

Length	Load	Load Sums	Moment Sums
200	Uniform Load = 3,000 pounds per foot	699.0	75733.50
201		702.0	76434.00
202		705.0	77137.50
203		708.0	77844.00
204		711.0	78553.50
205		714.0	79266.00
206		717.0	79981.50
207		720.0	80700.00
208		723.0	81421.50
209		726.0	82146.00
210		729.0	82873.50
211		732.0	83604.00
212		735.0	84337.50
213		738.0	85074.00
214		741.0	85813.50
215		744.0	86556.00
216		747.0	87301.50
217		750.0	88050.00
218		753.0	88801.50
219		756.0	89556.00
220		759.0	90313.50
221		762.0	91074.00
222		765.0	91837.50
223		768.0	92604.00
224		771.0	93373.50
225		774.0	94146.00
226		777.0	94921.50
227		780.0	95700.00
228		783.0	96481.50
229		786.0	97266.00
230		789.0	98053.50
231		792.0	98844.00
232		795.0	99637.50
233		798.0	100434.00
234		801.0	101233.50
235		804.0	102036.00
236		807.0	102841.50
237		810.0	103650.00
238		813.0	104461.50
239		816.0	105276.00
240		819.0	106093.50
241		822.0	106914.00
242		825.0	107737.50
243		828.0	108564.00
244		831.0	109393.50
245		834.0	110226.00
246		837.0	111061.50
247		840.0	111900.00
248		843.0	112741.50
249		846.0	113586.00
250		849.0	114433.50

Cooper's *E*60. 250′–300′

Length	Load	Load Sums	Moment Sums
250	Uniform Load = 3,000 pounds per foot	849.0	114433.50
251		852.0	115284.00
252		855.0	116137.50
253		858.0	116994.00
254		861.0	117853.50
255		864.0	118716.00
256		867.0	119581.50
257		870.0	120450.00
258		873.0	121321.50
259		876.0	122196.00
260		879.0	123073.50
261		882.0	123954.00
262		885.0	124837.50
263		888.0	125724.00
264		891.0	126613.50
265		894.0	127506.00
266		897.0	128401.50
267		900.0	129300.00
268		903.0	130201.50
269		906.0	131106.00
270		909.0	132013.50
271		912.0	132924.00
272		915.0	133837.50
273		918.0	134754.00
274		921.0	135673.50
275		924.0	136596.00
276		927.0	137521.50
277		930.0	138450.00
278		933.0	139381.50
279		936.0	140316.00
280		939.0	141253.50
281		942.0	142194.00
282		945.0	143137.50
283		948.0	144084.00
284		951.0	145033.50
285		954.0	145986.00
286		957.0	146941.50
287		960.0	147900.00
288		963.0	148861.50
289		966.0	149826.00
290		969.0	150793.50
291		972.0	151764.00
292		975.0	152737.50
293		978.0	153714.00
294		981.0	154693.50
295		984.0	155676.00
296		987.0	156661.50
297		990.0	157650.00
298		993.0	158641.50
299		996.0	159636.00
300		999.0	160633.50

Cooper's *E*60. 300'–350' Cooper's *E*60. 350'–400'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
300	Uniform Load = 3,000 pounds per foot	999.0	160633.50	350	Uniform Load = 3,000 pounds per foot	1149.0	214333.50
301		1002.0	161634.00	351		1152.0	215484.00
302		1005.0	162637.50	352		1155.0	216637.50
303		1008.0	163644.00	353		1158.0	217794.00
304		1011.0	164653.50	354		1161.0	218953.50
305		1014.0	165666.00	355		1164.0	220116.00
306		1017.0	166681.50	356		1167.0	221281.50
307		1020.0	167700.00	357		1170.0	222450.00
308		1023.0	168721.50	358		1173.0	223621.50
309		1026.0	169746.00	359		1176.0	224796.00
310		1029.0	170773.50	360		1179.0	225973.50
311		1032.0	171804.00	361		1182.0	227154.00
312		1035.0	172837.50	362		1185.0	228337.50
313		1038.0	173874.00	363		1188.0	229524.00
314		1041.0	174913.50	364		1191.0	230713.50
315		1044.0	175956.00	365		1194.0	231906.00
316		1047.0	177001.50	366		1197.0	233101.50
317		1050.0	178050.00	367		1200.0	234300.00
318		1053.0	179101.50	368		1203.0	235501.50
319		1056.0	180156.00	369		1206.0	236706.00
320		1059.0	181213.50	370		1209.0	237913.50
321		1062.0	182274.00	371		1212.0	239124.00
322		1065.0	183337.50	372		1215.0	240337.50
323		1068.0	184404.00	373		1218.0	241554.00
324		1071.0	185473.50	374		1221.0	242773.50
325		1074.0	186546.00	375		1224.0	243996.00
326		1077.0	187621.50	376		1227.0	245221.50
327		1080.0	188700.00	377		1230.0	246450.00
328		1083.0	189781.50	378		1233.0	247681.50
329		1086.0	190866.00	379		1236.0	248916.00
330		1089.0	191953.50	380		1239.0	250153.50
331		1092.0	193044.00	381		1242.0	251394.00
332		1095.0	194137.50	382		1245.0	252637.50
333		1098.0	195234.00	383		1248.0	253884.00
334		1101.0	196333.50	384		1251.0	255133.50
335		1104.0	197436.00	385		1254.0	256386.00
336		1107.0	198541.50	386		1257.0	257641.50
337		1110.0	199650.00	387		1260.0	258900.00
338		1113.0	200761.50	388		1263.0	260161.50
339		1116.0	201876.00	389		1266.0	261426.00
340		1119.0	202993.50	390		1269.0	262693.50
341		1122.0	204114.00	391		1272.0	263964.00
342		1125.0	205237.50	392		1275.0	265237.50
343		1128.0	206364.00	393		1278.0	266514.00
344		1131.0	207493.50	394		1281.0	267793.50
345		1134.0	208626.00	395		1284.0	269076.00
346		1137.0	209761.50	396		1287.0	270361.50
347		1140.0	210900.00	397		1290.0	271650.00
348		1143.0	212041.50	398		1293.0	272941.50
349		1146.0	213186.00	399		1296.0	274236.00
350		1149.0	214333.50	400		1299.0	275533.50

Common Standard 0′–50′					Common Standard 50′–100′				
Length	Wheel	Load	Load Sums	Moment Sums	Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	12.5	12.5	00.00	50	.....	....	.....	5120.00
1	.....	....	.....	12.50	51	.....	....	.....	5312.50
2	.....	....	.....	25.00	52	.....	....	.....	5505.00
3	.....	....	.....	37.50	53	.....	....	.....	5697.50
4	.....	....	.....	50.00	54	.....	....	.....	5890.00
5	.....	....	.....	62.50	55	.....	....	.....	6082.50
6	.....	....	.....	75.00	56	w. 10	12.5	205.0	6275.00
7	.....	....	.....	87.50	57	.....	....	.....	6480.00
8	w. 2	27.5	40.0	100.00	58	.....	....	.....	6685.00
9	.....	....	.....	140.00	59	.....	....	.....	6890.00
10	.....	....	.....	180.00	60	.....	....	.....	7095.00
11	.....	....	.....	220.00	61	.....	....	.....	7300.00
12	.....	....	.....	260.00	62	.....	....	.....	7505.00
13	w. 3	27.5	67.5	300.00	63	.....	....	.....	7710.00
14	.....	....	.....	367.50	64	w. 11	27.5	232.5	7915.00
15	.....	....	.....	435.00	65	.....	....	.....	8147.50
16	.....	....	.....	502.50	66	.....	....	.....	8380.00
17	.....	....	.....	570.00	67	.....	....	.....	8612.50
18	w. 4	27.5	95.0	637.50	68	.....	....	.....	8845.00
19	.....	....	.....	732.50	69	w. 12	27.5	260.0	9077.50
20	.....	....	.....	827.50	70	.....	....	.....	9337.50
21	.....	....	.....	922.50	71	.....	....	.....	9597.50
22	.....	....	.....	1017.50	72	.....	....	.....	9857.50
23	w. 5	27.5	122.5	1112.50	73	.....	....	.....	10117.50
24	.....	....	.....	1235.00	74	w. 13	27.5	287.5	10377.50
25	.....	....	.....	1357.50	75	.....	....	.....	10665.00
26	.....	....	.....	1480.00	76	.....	....	.....	10952.50
27	.....	....	.....	1602.50	77	.....	....	.....	11240.00
28	.....	....	.....	1725.00	78	.....	....	.....	11527.50
29	.....	....	.....	1847.50	79	w. 14	27.5	315.0	11815.00
30	.....	....	.....	1970.00	80	.....	....	.....	12130.00
31	.....	....	.....	2092.50	81	.....	....	.....	12445.00
32	w. 6	17.5	140.0	2215.00	82	.....	....	.....	12760.00
33	.....	....	.....	2355.00	83	.....	....	.....	13075.00
34	.....	....	.....	2495.00	84	.....	....	.....	13390.00
35	.....	....	.....	2635.00	85	.....	....	.....	13705.00
36	.....	....	.....	2775.00	86	.....	....	.....	14020.00
37	w. 7	17.5	157.5	2915.00	87	.....	....	.....	14335.00
38	.....	....	.....	3072.50	88	w. 15	17.5	332.5	14650.00
39	.....	....	.....	3230.00	89	.....	....	.....	14982.50
40	.....	....	.....	3387.50	90	.....	....	.....	15315.00
41	.....	....	.....	3545.00	91	.....	....	.....	15647.50
42	.....	....	.....	3702.50	92	.....	....	.....	15980.00
43	w. 8	17.5	175.0	3860.00	93	w. 16	17.5	350.0	16312.50
44	.....	....	.....	4035.00	94	.....	....	.....	16662.50
45	.....	....	.....	4210.00	95	.....	....	.....	17012.50
46	.....	....	.....	4385.00	96	.....	....	.....	17362.50
47	.....	....	.....	4560.00	97	.....	....	.....	17712.50
48	w. 9	17.5	192.5	4735.00	98	.....	....	.....	18062.50
49	.....	....	.....	4927.50	99	w. 17	17.5	367.5	18412.50
50	.....	....	.....	5120.00	100	.....	....	.....	18780.00

Common Standard 100'-150'

Length	Wheel	Load	Load Sums	Moment Sums
100				18780.00
101				19147.50
102				19515.00
103				19882.50
104	w. 18	17.5	385.0	20250.00
105				20635.00
106				21020.00
107				21405.00
108				21790.00
109			385.0	22175.00
110		Uniform Load = 2,500 pounds per foot	387.5	22561.25
111			390.0	22950.00
112			392.5	23341.25
113			395.0	23735.00
114			397.5	24131.25
115			400.0	24530.00
116			402.5	24931.25
117			405.0	25335.00
118			407.5	25741.25
119			410.0	26150.00
120			412.5	26561.25
121			415.0	26975.00
122			417.5	27391.25
123			420.0	27810.00
124			422.5	28231.25
125			425.0	28655.00
126			427.5	29081.25
127			430.0	29510.00
128			432.5	29941.25
129			435.0	30375.00
130			437.5	30811.25
131			440.0	31250.00
132			442.5	31691.25
133			445.0	32135.00
134			447.5	32581.25
135			450.0	33030.00
136			452.5	33481.25
137			455.0	33935.00
138			457.5	34391.25
139			460.0	34850.00
140			462.5	35311.25
141			465.0	35775.00
142			467.5	36241.25
143			470.0	36710.00
144			472.5	37181.25
145			475.0	37655.00
146			477.5	38131.25
147			480.0	38610.00
148			482.5	39091.25
149			485.0	39575.00
150			487.5	40061.25

Common Standard 150'-200'

Length	Load	Load Sums	Moment Sums
150	Uniform Load = 2,500 pounds per foot	487.5	40061.25
151		490.0	40550.00
152		492.5	41041.25
153		495.0	41535.00
154		497.5	42031.25
155		500.0	42530.00
156		502.5	43031.25
157		505.0	43535.00
158		507.5	44041.25
159		510.0	44550.00
160		512.5	45061.25
161		515.0	45575.00
162		517.5	46091.25
163		520.0	46610.00
164		522.5	47131.25
165		525.0	47655.00
166		527.5	48181.25
167		530.0	48710.00
168		532.5	49241.25
169		535.0	49775.00
170		537.5	50311.25
171		540.0	50850.00
172		542.5	51391.25
173		545.0	51935.00
174		547.5	52481.25
175		550.0	53030.00
176		552.5	53581.25
177		555.0	54135.00
178		557.5	54691.25
179		560.0	55250.00
180		562.5	55811.25
181		565.0	56375.00
182		567.5	56941.25
183		570.0	57510.00
184		572.5	58081.25
185		575.0	58655.00
186		577.5	59231.25
187		580.0	59810.00
188		582.5	60391.25
189		585.0	60975.00
190		587.5	61561.25
191		590.0	62150.00
192		592.5	62741.25
193		595.0	63335.00
194		597.5	63931.25
195		600.0	64530.00
196		602.5	65131.25
197		605.0	65735.00
198		607.5	66341.25
199		610.0	66950.00
200		612.5	67561.25

Common Standard 200′–250′				Common Standard 250′–300′			
Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
200	Uniform Load = 2,500 pounds per foot	612.5	67561.25	250	Uniform Load = 2,500 pounds per foot	737.5	101311.25
201		615.0	68175.00	251		740.0	102050.00
202		617.5	68791.25	252		742.5	102791.25
203		620.0	69410.00	253		745.0	103535.00
204		622.5	70031.25	254		747.5	104281.25
205		625.0	70655.00	255		750.0	105030.00
206		627.5	71281.25	256		752.5	105781.25
207		630.0	71910.00	257		755.0	106535.00
208		632.5	72541.25	258		757.5	107291.25
209		635.0	73175.00	259		760.0	108050.00
210		637.5	73811.25	260		762.5	108811.25
211		640.0	74450.00	261		765.0	109575.00
212		642.5	75091.25	262		767.5	110341.25
213		645.0	75735.00	263		770.0	111110.00
214		647.5	76381.25	264		772.5	111881.25
215		650.0	77030.00	265		775.0	112655.00
216		652.5	77681.25	266		777.5	113431.25
217		655.0	78335.00	267		780.0	114210.00
218		657.5	78991.25	268		782.5	114991.25
219		660.0	79650.00	269		785.0	115775.00
220		662.5	80311.25	270		787.5	116561.25
221		665.0	80975.00	271		790.0	117350.00
222		667.5	81641.25	272		792.5	118141.25
223		670.0	82310.00	273		795.0	118935.00
224		672.5	82981.25	274		797.5	119731.25
225		675.0	83655.00	275		800.0	120530.00
226		677.5	84331.25	276		802.5	121331.25
227		680.0	85010.00	277		805.0	122135.00
228		682.5	85691.25	278		807.5	122941.25
229		685.0	86375.00	279		810.0	123750.00
230		687.5	87061.25	280		812.5	124561.25
231		690.0	87750.00	281		815.0	125375.00
232		692.5	88441.25	282		817.5	126191.25
233		695.0	89135.00	283		820.0	127010.00
234		697.5	89831.25	284		822.5	127831.25
235		700.0	90530.00	285		825.0	128655.00
236		702.5	91231.25	286		827.5	129481.25
237		705.0	91935.00	287		830.0	130310.00
238		707.5	92641.25	288		832.5	131141.25
239		710.0	93350.00	289		835.0	131975.00
240		712.5	94061.25	290		837.5	132811.25
241		715.0	94775.00	291		840.0	133650.00
242		717.5	95491.25	292		842.5	134491.25
243		720.0	96210.00	293		845.0	135335.00
244		722.5	96931.25	294		847.5	136181.25
245		725.0	97655.00	295		850.0	137030.00
246		727.5	98381.25	296		852.5	137881.25
247		730.0	99110.00	297		855.0	138735.00
248		732.5	99841.25	298		857.5	139591.25
249		735.0	100575.00	299		860.0	140450.00
250		737.5	101311.25	300		862.5	141311.25

Common Standard 300'-350'				Common Standard 350'-400'			
Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
300	Uniform Load = 2,500 pounds per foot	862.5	141311.25	350	Uniform Load = 2,500 pounds per foot	987.50	187561.25
301		865.0	142175.00	351		990.00	188550.00
302		867.5	143041.25	352		992.50	189541.25
303		870.0	143910.00	353		995.00	190535.00
304		872.5	144781.25	354		997.50	191531.25
305		875.0	145655.00	355		1000.00	192530.00
306		877.5	146531.25	356		1002.50	193531.25
307		880.0	147410.00	357		1005.00	194535.00
308		882.5	148291.25	358		1007.50	195541.25
309		885.0	149175.00	359		1010.00	196550.00
310		887.5	150061.25	360		1012.50	197561.25
311		890.0	150950.00	361		1015.00	198575.00
312		892.5	151841.25	362		1017.50	199591.25
313		895.0	152735.00	363		1020.00	200610.00
314		897.5	153631.25	364		1022.50	201631.25
315		900.0	154530.00	365		1025.00	202655.00
316		902.5	155431.25	366		1027.50	203681.25
317		905.0	156335.00	367		1030.00	204710.00
318		907.5	157241.25	368		1032.50	205741.25
319		910.0	158150.00	369		1035.00	206775.00
320		912.5	159061.25	370		1037.50	207811.25
321		915.0	159975.00	371		1040.00	208850.00
322		917.5	160891.25	372		1042.50	209891.25
323		920.0	161810.00	373		1045.00	210935.00
324		922.5	162731.25	374		1047.50	211981.25
325		925.0	163655.00	375		1050.00	213030.00
326		927.5	164581.25	376		1052.50	214081.25
327		930.0	165510.00	377		1055.00	215135.00
328		932.5	166441.25	378		1057.50	216191.25
329		935.0	167375.00	379		1060.00	217250.00
330		937.5	168311.25	380		1062.50	218311.25
331		940.0	169250.00	381		1065.00	219375.00
332		942.5	170191.25	382		1067.50	220441.25
333		945.0	171135.00	383		1070.00	221510.00
334		947.5	172081.25	384		1072.50	222581.25
335		950.0	173030.00	385		1075.00	223655.00
336		952.5	173981.25	386		1077.50	224731.25
337		955.0	174935.00	387		1080.00	225810.00
338		957.5	175891.25	388		1082.50	226891.25
339		960.0	176850.00	389		1085.00	227975.00
340		962.5	177811.25	390		1087.50	229061.25
341		965.0	178775.00	391		1090.00	230150.00
342		967.5	179741.25	392		1092.50	231241.25
343		970.0	180710.00	393		1095.00	232335.00
344		972.5	181681.25	394		1097.50	233431.25
345		975.0	182655.00	395		1100.00	234530.00
346		977.5	183631.25	396		1102.50	235631.25
347		980.0	184610.00	397		1105.00	236735.00
348		982.5	185591.25	398		1107.50	237841.25
349		985.0	186575.00	399		1110.00	238950.00
350		987.5	187561.25	400		1112.50	240061.25

Lackawanna 0′–50′					Lackawanna 50′–100′				
Length	Wheel	Load	Load Sums	Moment Sums	Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	11	11.00	00.000	50				4744.000
1				11.000	51				4911.000
2				22.000	52				5078.000
3				33.000	53				5245.000
4				44.000	54	w. 10	11	178.00	5412.000
5				55.000	55				5590.000
6				66.000	56				5768.000
7	w. 2	25	36.00	77.000	57				5946.000
8				113.000	58				6124.000
9				149.000	59				6302.000
10				185.000	60				6480.000
11				221.000	61	w. 11	25	203.00	6658.000
12	w. 3	25	61.00	257.000	62				6861.000
13				318.000	63				7064.000
14				379.000	64				7267.000
15				440.000	65				7470.000
16				501.000	66	w. 12	25	228.00	7673.000
17	w. 4	25	86.00	562.000	67				7901.000
18				648.000	68				8129.000
19				734.000	69				8357.000
20				820.000	70				8585.000
21				906.000	71	w. 13	25	253.00	8813.000
22	w. 5	25	111.00	992.000	72				9066.000
23				1103.000	73				9319.000
24				1214.000	74				9572.000
25				1325.000	75				9825.000
26				1436.000	76	w. 14	25	278.00	10078.000
27				1547.000	77				10356.000
28				1658.000	78				10634.000
29				1769.000	79				10912.000
30				1880.000	80				11190.000
31	w. 6	14	125.00	1991.000	81				11468.000
32				2116.000	82				11746.000
33				2241.000	83				12024.000
34				2366.000	84				12302.000
35				2491.000	85	w. 15	14	292.00	12580.000
36	w. 7	14	139.00	2616.000	86				12872.000
37				2755.000	87				13146.000
38				2894.000	88				13456.000
39				3033.000	89				13748.000
40				3172.000	90	w. 16	14	306.00	14040.000
41	w. 8	14	153.00	3311.000	91				14346.000
42				3464.000	92				14652.000
43				3617.000	93				14958.000
44				3770.000	94				15264.000
45				3923.000	95	w. 17	14	320.00	15570.000
46	w. 9	14	167.00	4076.000	96				15890.000
47				4243.000	97				16210.000
48				4410.000	98				16530.000
49				4577.000	99				16850.000
50				4744.000	100	w. 18	14	334.00	17170.000

Lackawanna 100′-150′					Lackawanna 150′-200′			
Length	Wheel	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
100	w. 18	14	334.00	17170.000	150		437.50	36250.500
101	......	..	......	17504.000	151		439.75	36689.125
102	......	..	......	17838.000	152		442.00	37130.000
103	......	..	......	18172.000	153		444.25	37573.125
104			334.00	18506.000	154		446.50	38018.500
105			336.25	18841.125	155		448.75	38466.125
106			338.50	19178.500	156		451.00	38916.000
107			340.75	19518.125	157		453.25	39368.125
108			343.00	19860.000	158		455.50	39822.500
109			345.25	20204.125	159		457.75	40279.125
110			347.50	20550.500	160		460.00	40738.000
111			349.75	20899.125	161		462.25	41199.125
112			352.00	21250.000	162		464.50	41662.500
113			354.25	21603.125	163		466.75	42128.125
114			356.50	21958.500	164		469.00	42596.000
115			358.75	22316.125	165		471.25	43066.125
116			361.00	22676.000	166		473.50	43538.500
117			363.25	23038.125	167		475.75	44013.125
118			365.50	23402.500	168		478.00	44490.000
119			367.75	23769.125	169		480.25	44969.125
120			370.00	24138.000	170		482.50	45450.500
121			372.25	24509.125	171		484.75	45934.125
122			374.50	24882.500	172		487.00	46420.000
123			376.75	25258.125	173		489.25	46908.125
124			379.00	25636.000	174		491.50	47398.500
125		Uniform Load = 2,250 pounds per foot	381.25	26016.125	175	Uniform Load = 2,250 pounds per foot	493.75	47891.125
126			383.50	26398.500	176		496.00	48386.000
127			385.75	26783.125	177		498.25	48883.125
128			388.00	27170.000	178		500.50	49382.500
129			390.25	27559.125	179		502.75	49884.125
130			392.50	27950.500	180		505.00	50388.000
131			394.75	28344.125	181		507.25	50894.125
132			397.00	28740.000	182		509.50	51402.500
133			399.25	29138.125	183		511.75	51913.125
134			401.50	29538.500	184		514.00	52426.000
135			403.75	29941.125	185		516.25	52941.125
136			406.00	30346.000	186		518.50	53458.500
137			408.25	30753.125	187		520.75	53978.125
138			410.50	31162.500	188		523.00	54500.000
139			412.75	31574.125	189		525.25	55024.125
140			415.00	31988.000	190		527.50	55550.500
141			417.25	32404.125	191		529.75	56079.125
142			419.50	32882.500	192		532.00	56610.000
143			421.75	33243.125	193		534.25	57143.125
144			424.00	33666.000	194		536.50	57678.500
145			426.25	34091.125	195		538.75	58216.125
146			428.50	34518.500	196		541.00	58756.000
147			430.75	34948.125	197		543.25	59298.125
148			433.00	35380.000	198		545.50	59842.500
149			435.25	35814.125	199		547.75	60389.125
150			437.50	36250.500	200		550.00	60938.000

LACKAWANNA 200′–250′

Length	Load	Load Sums	Moment Sums
200	Uniform Load = 2,250 pounds per foot	550.00	60938.000
201		552.25	61489.125
202		554.50	62042.500
203		556.75	62598.125
204		559.00	63156.000
205		561.25	63716.125
206		563.50	64278.500
207		565.75	64843.125
208		568.00	65410.000
209		570.25	65979.125
210		572.50	66550.500
211		574.75	67124.125
212		577.00	67700.000
213		579.25	68278.125
214		581.50	68858.500
215		583.75	69441.125
216		586.00	70026.000
217		588.25	70613.125
218		590.50	71202.500
219		592.75	71794.125
220		595.00	72388.000
221		597.25	72984.125
222		599.50	73582.500
223		601.75	74183.125
224		604.00	74786.000
225		606.25	75391.125
226		608.50	75998.500
227		610.75	76608.125
228		613.00	77220.000
229		615.25	77834.125
230		617.50	78450.500
231		619.75	79069.125
232		622.00	79690.000
233		624.25	80313.125
234		626.50	80938.500
235		628.75	81566.125
236		631.00	82196.000
237		633.25	82828.125
238		635.50	83462.500
239		637.75	84099.125
240		640.00	84738.000
241		642.25	85379.125
242		644.50	86022.500
243		646.75	86668.125
244		649.00	87316.000
245		651.25	87966.125
246		653.50	88618.500
247		655.75	89273.125
248		658.00	89930.000
249		660.25	90589.125
250		662.50	91250.500

LACKAWANNA 250′–300′

Length	Load	Load Sums	Moment Sums
250	Uniform Load = 2,250 pounds per foot	662.50	91250.500
251		664.75	91914.125
252		667.00	92580.000
253		669.25	93248.125
254		671.50	93918.500
255		673.75	94591.125
256		676.00	95266.000
257		678.25	95943.125
258		680.50	96622.500
259		682.75	97304.125
260		685.00	97988.000
261		687.25	98674.125
262		689.50	99362.500
263		691.75	100053.125
264		694.00	100746.000
265		696.25	101441.125
266		698.50	102138.500
267		700.75	102838.125
268		703.00	103540.000
269		705.25	104244.125
270		707.50	105950.500
271		709.75	105659.125
272		712.00	106370.000
273		714.25	107083.125
274		716.50	107798.500
275		718.75	108516.125
276		721.00	109236.000
277		723.25	109958.125
278		725.50	110682.500
279		727.75	111409.125
280		730.00	112138.000
281		732.25	112869.125
282		734.50	113602.500
283		736.75	114338.125
284		739.00	115076.000
285		741.25	115816.125
286		743.50	116558.500
287		745.75	117303.125
288		748.00	118050.000
289		750.25	118799.125
290		752.50	119550.500
291		754.75	120304.125
292		757.00	121060.000
293		759.25	121818.125
294		761.50	122578.500
295		763.75	123341.125
296		766.00	124106.000
297		768.25	124873.125
298		770.50	125642.500
299		772.75	126414.125
300		775.00	127188.000

Lackawanna 300′–350′

Length	Load	Load Sums	Moment Sums
300	Uniform Load = 2,250 pounds per foot	775.00	127188.000
301		777.25	127964.125
302		779.50	128742.500
303		781.75	129523.125
304		784.00	130306.000
305		786.25	131091.125
306		788.50	131878.500
307		790.75	132668.125
308		793.00	133460.000
309		795.25	134254.125
310		797.50	135050.500
311		799.75	135849.125
312		802.00	136650.000
313		804.25	137453.125
314		806.50	138258.500
315		808.75	139066.125
316		811.00	139876.000
317		813.25	140688.125
318		815.50	141502.500
319		817.75	142319.125
320		820.00	143138.000
321		822.25	143959.125
322		824.50	144782.500
323		826.75	145608.125
324		829.00	146436.000
325		831.25	147266.125
326		833.50	148098.500
327		835.75	148933.125
328		838.00	149770.000
329		840.25	150609.125
330		842.50	151450.500
331		844.75	152294.125
332		847.00	153140.000
333		849.25	153988.125
334		851.50	154838.500
335		853.75	155691.125
336		856.00	156546.000
337		858.25	157403.125
338		860.50	158262.500
339		862.75	159124.125
340		865.00	159988.000
341		867.25	160854.125
342		869.50	161722.500
343		871.75	162593.125
344		874.00	163466.000
345		876.25	164341.125
346		878.50	165218.500
347		880.75	166098.125
348		883.00	166980.000
349		885.25	167864.125
350		887.50	168750.500

Lackawanna 350′–400′

Length	Load	Load Sums	Moment Sums
350	Uniform Load = 2,250 pounds per foot	887.50	168750.500
351		889.75	169639.125
352		892.00	170530.000
353		894.25	171423.125
354		896.50	172318.500
355		898.75	173216.125
356		901.00	174116.000
357		903.25	175018.125
358		905.50	175922.500
359		907.75	176829.125
360		910.00	177738.000
361		912.25	178649.125
362		914.50	179562.500
363		916.75	180478.125
364		919.00	181396.000
365		921.25	182316.125
366		923.50	183238.500
367		925.75	184163.125
368		928.00	185090.000
369		930.25	186019.125
370		932.50	186950.500
371		934.75	187884.125
372		937.00	188820.000
373		939.25	189758.125
374		941.50	190698.500
375		943.75	191641.125
376		946.00	192586.000
377		948.25	193533.125
378		950.50	194482.500
379		952.75	195434.125
380		955.00	196388.000
381		957.25	197344.125
382		959.50	198302.500
383		961.75	199263.125
384		964.00	200226.000
385		966.25	201191.125
386		968.50	202158.500
387		970.75	203128.125
388		973.00	204100.000
389		975.25	205074.125
390		977.50	206050.500
391		979.75	207029.125
392		982.00	208010.000
393		984.25	208993.125
394		986.50	209978.500
395		988.75	210966.125
396		991.00	211956.000
397		993.25	212948.125
398		995.50	213942.500
399		997.75	214939.125
400		1000.00	215938.000

TABLE 3

Position of Cooper's Loadings for Maximum Stress

Shorter Segment l_1

Segments (Longer Segment l_2)	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100	110	120	130	140
300–260	2	2	3	3	4	4	5	5	6	7	7	8	9	10	10	11	11	12	12	13	14	15	17	18
250–200	2	2	3	3	4	4	5	5	6	7	8	8	9	10	11	11	12	12	12	13	14	15	17	18
190–150	2	2	3	3	4	4	5	5	6	7	8	9	9	11	11	12	12	12	12	13	14	15	17	18
140	2	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	12	12	12	13	14	15	17	18
130	2	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	12	12	12	13	14	15	17	..
120	2	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	12	13	13	13	14	15	..	..
110	2	3	3	3	4	4	5	6	7	7	8	9	10	11	12	12	12	13	13	13	14	..	..	..
100	2	3	3	3	4	5	5	6	14	14	14	13	13	11	12	12	12	13	13	13	..	..	..	..
95	2	3	3	4	4	5	13	13	13	13	13	13	13	13	12	12	12	13	13	..	..	..	..	..
90	2	3	3	4	4	5	13	13	13	13	13	13	13	13	12	12	12	13	..	..	..	..	..	..
85	2	3	3	4	4	5	13	13	12	13	13	12	13	13	12	12	12	..	..	..	..	..	..	..
80	2	3	3	4	4	13	13	13	12	12	12	12	12	12	12	12	..	..	..	..	..	..	..	..
75	2	3	3	4	4	13	13	12	12	12	12	12	12	12	12	..	..	..	..	..	..	..	..	..
70	2	3	3	4	4	13	13	12	12	12	12	11	11	11	..	..	..	..	..	..	..	..	..	..
65	2	3	3	4	4	12	12	12	12	12	11	11	11	..	..	..	..	..	..	..	..	..	..	..
60	11	3	3	4	4	5	13	12	11	11	11	11	..	..	..	..	..	..	..	..	..	..	..	..
55	11	12	12	12	4	12	13	12	12	13	11	..	..	..	..	..	..	..	..	..	..	..	..	..
50	11	12	12	12	12	12	13	13	13	12	..	..	..	..	..	..	..	..	..	..	..	..	..	..
45	2	3	12	12	12	12	13	13	13	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
40	2	3	3	3	12	12	13	13	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
35	2	3	3	4	4	13	13	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
30	2	3	3	4	4	13	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
25	2	3	3	4	4	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
20	2	4	3	4	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
15	2	3	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
10	2	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
5	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..

General Notes.—The table gives wheel for maximum for any stress which has a triangular influence line.

In case of two unequal segments, the live load approaches on the longer segment except where wheel is overlined, when live load approaches on shorter segment.

When both segments are each greater than 142 ft., advance load on longer segment first, and upon next segment until wheel No. 1 is within 33 feet of the far end of the latter.

TABLE 4

Position of Cooper's Loadings for Absolute Maximum Bending Moment in Girder Bridges Without Panels

S = Span in feet.

c = Distance in feet that wheel No. 1 has moved to left beyond centre of span.

w = wheel under which absolute maximum bending moment occurs.

a = distance that w is to left from centre of span.

b = " " w " right " " " "

S	c	w	a	b
0′ to 8′.5	8′.00	2	0′.00	....
8.5 " 11.1	9.25	2	1.25	....
11.1 " 18.7	13.00	3	0.00	....
18.7 " 27.6	14.25	3	1.25	....
27.6 " 34.9	13.39	3	0.39	....
34.9 " 38.7	17.06	4	....	0.94
38.7 " 48.6	18.21	4	0.21	....
48.6 " 53.7	19.45	4	1.45	....
53.7 " 58.4	74.13	13	0.13	....
58.4 " 63.2	75.37	13	1.37	....
63.2 " 70.00	74.07	13	0.07	....

Note.—For spans greater than 70 feet, the maximum centre moment equals the absolute maximum bending moment with an error of less than one per cent.

TABLE 5

Position of Cooper's Loadings for Maximum End Shear in Girder Bridges Without Panels

Span	Direction Load Moves	Position of Load	Location of Maximum Shear
0′ to 23′	Right to left	w_2 at left end	Left end
23 " 27	Right to left	w_4 at right end	Right end
27 " 46	Right to left	w_2 at left end	Left end
46 " 62	Right to left	w_{11} at left end	Left end
62 " 400	Right to left	w_2 at left end	Left end

TABLE 6

Position of Cooper's Loadings for Maximum Shear in Panels of Girder and Truss Bridges

Number of Panels	Panel	Panel Length in Feet													
		22	23	24	25	26	27	28	29	30	31	32	33	34	35
6	0–1	4	4	4	4	4	4	4	4	4	4	5	5	5	5
	1–2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
	2–3	3	3	3	3	3	3	3	3	3	3	3	3	3	4
	3–4	2	2	2	2	2	2	2	2	2	3	3	3	3	3
	4–5	2	2	2	2	2	2	2	2	2	2	2	2	2	2
7	0–1	4	4	4	4	4	4	4	4	4	4	4	5	5	5
	1–2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
	2–3	3	3	3	3	3	3	3	3	3	3	3	3	4	4
	3–4	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	4–5	2	2	2	2	2	2	2	2	2	2	2	2	3	3
	5–6	2	2	2	2	2	2	2	2	2	2	2	2	2	2
8	0–1	3	4	4	4	4	4	4	4	4	4	4	5	5	5
	1–2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
	2–3	3	3	3	3	3	3	3	3	3	4	4	4	4	4
	3–4	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	4–5	2	2	2	2	2	3	3	3	3	3	3	3	3	3
	5–6	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	6–7	2	2	2	2	2	2	2	2	2	2	2	2	2	2
9	0–1	3	4	4	4	4	4	4	4	4	4	4	4	5	5
	1–2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
	2–3	3	3	3	3	3	3	3	3	4	4	4	4	4	4
	3–4	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	4–5	2	3	3	3	3	3	3	3	3	3	3	3	3	3
	5–6	2	2	2	2	2	2	2	2	2	3	3	3	3	3
	6–7	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	7–8	2	2	2	2	2	2	2	2	2	2	2	2	2	2
10	0–1	3	4	4	4	4	4	4	4	4	4	4	4	5	5
	1–2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
	2–3	3	3	3	3	3	3	3	3	4	4	4	4	4	4
	3–4	3	3	3	3	3	3	3	3	3	3	3	3	3	4
	4–5	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	5–6	2	2	2	2	2	2	3	3	3	3	3	3	3	3
	6–7	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	7–8	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	8–9	1	1	1	1	1	1	1	1	2	2	2	2	2	2

NOTE.—Place tabulated wheel at right end of corresponding panel with locomotive advancing toward left.

TABLE 7

Maximum Moments, Shears, and Pier Reactions for Cooper's Standard Loadings

(Figures for One Rail)

Span	E40 Max. Moment	E40 Max. Shears End	E40 Max. Shears ¼ Pt.	E40 Max. Shears Cent.	E40 Max. Pier React.	E50 Max. Moment	E50 Max. Shears End	E50 Max. Shears ¼ Pt.	E50 Max. Shears Cent.	E50 Max. Pier React.
10......	56.3	30.0	20.0	10.0	40.0	70.4	37.5	25.0	12.5	50.0
11......	65.7	32.7	20.9	10.9	43.7	82.1	40.9	26.1	13.6	54.5
12......	80.0	35.0	21.7	11.7	46.7	100.0	43.8	27.1	14.6	58.4
13......	95.0	36.9	22.3	12.3	49.2	118.8	46.2	27.9	15.4	61.6
14......	110.0	38.6	23.6	12.9	52.2	137.5	48.2	29.5	16.2	65.2
15......	125.0	40.0	25.0	13.3	54.7	156.3	50.0	31.3	16.6	68.3
16......	140.0	42.5	26.3	13.7	56.9	175.0	53.1	32.9	17.1	71.1
17......	155.0	44.7	27.4	13.8	58.8	193.8	55.9	34.3	17.3	73.5
18......	170.0	46.7	28.3	13.9	60.7	212.5	58.3	35.4	17.4	75.9
19......	186.6	48.4	29.2	14.0	62.9	233.3	60.5	36.5	17.5	78.6
20......	206.3	50.0	30.0	14.0	65.6	257.9	62.5	37.5	17.5	81.9
21......	226.0	51.4	31.4	14.5	68.0	282.5	64.3	39.2	18.1	84.9
22......	245.7	52.7	32.7	15.0	70.2	307.1	65.9	40.9	18.8	87.6
23......	265.4	53.9	33.9	15.4	72.2	331.8	67.4	42.4	19.3	90.2
24......	285.2	55.4	35.0	15.8	74.0	356.5	69.3	43.8	19.8	92.4
25......	305.0	56.8	36.0	16.2	75.7	381.3	71.0	45.0	20.2	94.6
26......	324.8	58.1	36.9	16.5	77.7	406.0	72.6	46.1	20.6	97.1
27......	344.6	59.2	37.8	16.9	80.2	430.8	74.0	47.2	21.1	100.1
28......	365.5	60.4	38.6	17.1	82.3	456.9	75.5	48.2	21.4	102.8
29......	388.0	61.6	39.3	17.4	84.4	485.0	76.9	49.1	21.8	105.4
30......	410.5	63.0	40.0	17.7	86.3	513.0	78.8	50.0	22.1	107.9
31......	432.9	64.4	40.7	18.2	88.5	541.1	80.5	50.9	22.7	110.6
32......	455.4	65.7	41.3	18.8	91.0	569.3	82.1	51.8	23.4	113.7
33......	477.9	66.9	42.0	19.2	93.3	597.4	83.7	52.5	24.0	116.7
34......	500.6	68.1	42.8	19.7	95.5	625.8	85.1	53.5	24.6	119.4
35......	523.0	69.2	43.5	20.1	97.5	653.8	86.5	54.4	25.1	122.0
36......	548.6	70.6	44.1	20.6	99.6	685.8	88.2	55.1	25.8	124.4
37......	574.3	71.9	44.8	21.0	101.5	717.9	89.8	56.0	26.2	126.9
38......	600.0	73.1	45.4	21.3	103.7	750.0	91.4	56.7	26.6	129.7
39......	626.6	74.3	46.0	21.7	105.9	783.3	92.9	57.5	27.1	132.3
40......	655.6	75.4	46.8	22.0	108.0	819.5	94.3	58.5	27.5	135.0
41......	684.6	76.8	47.5	22.3	110.0	855.8	96.0	59.4	27.9	137.6
42......	713.6	78.4	48.2	22.6	112.1	892.0	97.6	60.2	28.3	140.2
43......	742.6	79.4	48.9	22.9	114.3	928.3	99.2	61.1	28.6	142.9
44......	771.6	80.6	49.5	23.2	116.5	964.5	100.7	61.9	29.0	145.6
45......	800.6	81.7	50.1	23.4	118.6	1000.8	102.1	62.6	29.3	148.3
46......	829.8	82.8	50.7	23.7	120.7	1037.3	103.5	63.4	29.6	150.9
47......	858.6	83.8	51.4	23.9	122.7	1073.3	104.9	64.2	29.9	153.4
48......	887.6	85.0	52.1	24.2	124.8	1109.5	106.3	65.1	30.2	156.0
49......	918.8	86.1	52.8	24.5	126.8	1148.5	107.7	66.0	30.6	158.5
50......	950.9	87.2	53.5	24.9	128.7	1188.6	109.0	66.8	31.1	161.0
51......	983.1	88.4	54.1	25.2	131.0	1228.9	110.4	67.6	31.5	163.6
52......	1015.2	89.3	54.8	25.5	133.3	1269.0	111.8	68.5	31.9	166.6
53......	1047.4	90.5	55.4	25.8	135.6	1309.2	113.1	69.2	32.3	169.6

TABLE 7.—*Continued*

Maximum Moments, Shears, and Pier Reactions for Cooper's Standard Loadings

(Figures for One Rail)

Span	*E*40					*E*50				
	Max. Moment	Max. Shears			Max. Pier React.	Max. Moment	Max. Shears			Max. Pier React.
		End	¼ Pt.	Cent.			End	¼ Pt.	Cent.	
54......	1081.4	91.5	56.1	26.1	138.0	1351.8	114.5	70.1	32.6	172.5
55......	1116.9	92.6	56.8	26.4	140.3	1396.1	115.8	71.0	33.0	175.4
56......	1152.4	93.7	57.5	26.6	142.7	1440.5	117.2	71.8	33.3	178.5
57......	1187.9	94.8	58.2	26.9	145.4	1484.9	118.5	72.7	33.6	181.8
58......	1223.4	95.9	58.8	27.2	148.1	1529.2	119.8	73.5	34.0	185.1
59......	1261.0	97.0	59.5	27.5	150.6	1576.2	121.2	74.4	34.4	188.4
60......	1299.6	98.0	60.1	27.9	153.2	1624.5	122.5	75.2	34.9	191.5
61......	1338.3	99.2	60.7	28.2	155.7	1672.9	123.9	76.0	35.2	194.7
62......	1377.0	100.1	61.3	28.5	158.2	1721.2	125.2	76.6	35.6	197.7
63......	1415.6	101.3	61.8	28.8	160.4	1769.5	126.6	77.4	36.0	200.7
64......	1455.5	102.6	62.4	29.1	162.6	1819.4	128.2	78.0	36.4	203.6
65......	1497.5	103.8	63.0	29.4	165.2	1871.9	129.7	78.8	36.8	206.7
66......	1539.5	105.0	63.6	29.7	167.8	1924.4	131.2	79.5	37.1	209.7
67......	1581.5	106.4	64.2	30.0	170.1	1976.9	133.0	80.3	37.5	212.7
68......	1623.5	107.8	64.8	30.2	172.5	2029.4	134.8	81.0	37.8	215.6
69......	1665.5	109.2	65.4	30.5	174.8	2081.9	136.5	81.7	38.1	218.5
70......	1707.5	110.5	65.9	30.7	177.1	2134.4	138.1	82.4	38.4	221.3
71......	1749.3	111.8	66.5	31.1	179.3	2186.6	139.8	83.1	38.8	224.1
72......	1793.0	113.3	67.0	31.4	181.5	2241.2	141.7	83.8	39.2	226.9
73......	1833.9	114.8	67.5	31.7	183.7	2292.4	143.5	84.4	39.6	229.6
74......	1879.2	116.3	68.0	32.0	186.0	2349.0	145.3	85.0	40.0	232.4
75......	1925.8	117.7	68.6	32.3	188.2	2407.3	147.1	85.7	40.4	235.2
76......	1972.0	119.1	69.2	32.6	190.4	2465.0	148.8	86.5	40.8	238.0
77......	2019.1	120.4	69.9	32.9	192.5	2523.9	150.5	87.4	41.1	240.7
78......	2065.0	121.7	70.5	33.2	194.7	2581.2	152.1	88.2	41.5	243.3
79......	2112.3	123.0	71.1	33.4	196.8	2640.4	153.8	88.9	41.7	245.9
80......	2160.5	124.2	71.7	33.7	198.9	2700.6	155.3	89.6	42.1	248.6
81......	2207.7	125.6	72.3	34.0	200.9	2759.6	157.0	90.4	42.5	251.1
82......	2256.7	126.9	73.0	34.4	203.0	2820.9	158.6	91.2	43.0	253.6
83......	2306.5	128.2	73.7	34.7	205.0	2883.1	160.3	92.1	43.4	256.1
84......	2356.3	129.5	74.4	35.0	206.9	2945.4	161.8	93.0	43.7	258.7
85......	2406.9	130.7	75.1	35.3	208.9	3008.6	163.4	93.9	44.1	260.8
86......	2459.6	132.1	75.8	35.6	210.8	3074.5	165.1	94.3	44.5	263.0
87......	2510.6	133.4	76.5	35.9	212.8	3138.3	166.8	95.7	44.9	265.6
88......	2564.2	134.7	77.1	36.2	214.7	3205.3	168.4	96.5	45.2	268.3
89......	2615.9	136.0	77.9	36.5	216.7	3269.9	170.0	97.4	45.6	270.8
90......	2670.5	137.2	78.7	36.7	218.6	3338.1	171.5	98.4	45.9	273.2
91......	2723.0	138.5	79.5	37.0	220.6	3403.7	173.1	99.4	46.2	275.6
92......	2776.7	139.8	80.3	37.3	222.5	3470.9	174.7	100.4	46.6	278.0
93......	2831.5	141.1	81.0	37.5	224.4	3539.3	176.4	101.2	46.9	280.3
94......	2885.3	142.4	81.7	37.8	226.3	3606.6	178.0	102.1	47.3	282.7
95......	2939.5	143.6	82.5	38.0	228.1	3674.3	179.5	103.1	47.5	285.1
96......	2994.5	144.8	83.3	38.3	230.0	3743.1	181.0	104.1	47.9	287.5
97......	3049.0	146.2	84.2	38.5	231.8	3811.2	182.7	105.1	48.1	289.7

TABLE 7.—*Continued*

Maximum Moments, Shears and Pier Reactions for Cooper's Standard Loadings

(Figures for One Rail)

Span	*E40* Max. Moment	*E40* Max. Shears End	*E40* Max. Shears ¼ Pt.	*E40* Max. Shears Cent.	*E40* Max. Pier React.	*E50* Max. Moment	*E50* Max. Shears End	*E50* Max. Shears ¼ Pt.	*E50* Max. Shears Cent.	*E50* Max. Pier React.
98......	3106.5	147.5	85.0	38.8	233.6	3883.1	184.3	106.2	48.5	292.0
99......	3162.3	148.8	85.8	39.1	235.4	3952.9	186.0	107.2	48.9	294.2
100......	3219.9	150.0	86.6	39.4	237.2	4024.9	187.5	108.2	49.2	296.5
101......	3277.6	151.2	87.3	39.6	238.9	4097.0	189.0	109.1	49.5	298.6
102......	3335.9	152.4	88.1	39.9	240.6	4169.9	190.6	110.1	49.9	300.8
103......	3410.6	153.7	88.8	40.1	242.4	4263.3	192.1	111.0	50.1	303.0
104......	3475.2	154.9	89.5	40.4	244.2	4344.0	193.6	111.9	50.5	305.3
105......	3537.6	156.1	90.3	40.6	246.0	4422.0	195.1	112.7	50.7	307.5
106......	3600.3	157.3	90.9	40.9	247.8	4500.4	196.6	113.6	51.1	309.8
107......	3666.6	158.5	91.7	41.1	249.6	4583.3	198.1	114.5	51.5	312.0
108......	3745.3	159.6	92.4	41.3	251.4	4681.6	199.5	115.5	51.7	314.2
109......	3818.4	160.8	93.2	41.6	253.1	4773.0	201.0	116.4	52.0	316.3
110......	3886.8	162.0	93.9	41.8	254.8	4858.5	202.5	117.4	52.3	318.5
111......	3958.2	163.2	94.6	42.0	256.5	4947.7	204.0	118.2	52.5	320.7
112......	4026.9	164.4	95.3	42.2	258.2	5033.6	205.5	119.1	52.7	322.8
113......	4099.0	165.5	96.0	42.5	259.9	5123.8	207.0	120.0	53.1	324.9
114......	4172.0	166.7	96.8	42.8	261.6	5215.0	208.4	121.0	53.5	327.0
115......	4245.0	167.9	97.5	43.1	263.3	5306.2	209.9	121.9	53.9	329.0
116......	4318.8	169.0	98.3	43.4	264.9	5398.5	211.3	122.9	54.2	331.1
117......	4389.5	170.2	99.0	43.7	266.7	5486.9	212.8	123.7	54.6	333.3
118......	4463.8	171.4	99.7	43.9	268.5	5579.7	214.2	124.6	54.9	335.6
119......	4538.8	172.5	100.4	44.2	270.2	5673.5	215.7	125.5	55.3	337.8
120......	4614.1	173.7	101.1	44.5	272.0	5767.6	217.1	126.4	55.6	340.0
121......	4686.5	174.8	101.8	44.7	273.8	5858.1	218.6	127.2	55.9	342.2
122......	4762.7	176.0	102.5	45.0	275.6	5953.4	220.0	128.1	56.2	344.5
123......	4836.2	177.1	103.2	45.3	277.4	6045.2	221.4	129.0	56.5	346.7
124......	4917.4	178.3	104.0	45.7	279.2	6146.7	222.8	130.0	57.0	349.0
125......	4996.4	179.4	104.7	46.0	281.0	6245.5	224.2	130.9	57.5	351.2
150......	7062.3	207.4	121.8	54.4	325.4	8827.9	259.2	152.2	68.0	406.7
175......	9352.5	234.5	138.3	62.5	371.7	11690.6	293.1	172.9	78.2	464.6
200......	11873.0	261.0	153.4	70.4	419.0	14841.2	326.3	191.8	88.0	523.8
250......	17592.5	313.2	183.7	85.0	515.2	21990.6	391.5	229.6	106.3	644.0

Note.—Moments are given in thousand foot-pounds.

Shears are given in thousand pounds.

Pier reactions are given in thousand pounds and are for piers between two spans each equal to the tabulated span.

TABLE 8

Maximum Moments for Truss Bridges—Cooper's *E*50 for One Rail

Moments Given in Thousands of Foot-Pounds

Panel Points 0 1 2 3 4 5 6 7 8 9

Panels in Truss	Panel Points	Panel Lengths											
		8′ 0″	8′ 6″	9′ 0″	9′ 6″	10′ 0″	10′ 6″	11′ 0″	11′ 6″	12′ 0″	12′ 6″	13′ 0″	13′ 6″
3	1	325	359	392	425	464	503	541	580	619	661	707	755
4	1	433	483	533	582	632	688	743	799	859	918	982	1046
	2	569	625	683	747	819	892	964	1037	1110	1189	1269	1352
5	1	540	599	662	728	794	861	930	1001	1071	1140	1217	1298
	2	790	877	964	1051	1149	1255	1361	1468	1574	1675	1792	1910
6	1	641	710	784	859	937	1017	1100	1186	1280	1375	1485	1600
	2	1008	1115	1228	1347	1466	1587	1719	1857	1997	2135	2289	2451
	3	1109	1221	1351	1484	1618	1767	1925	2070	2240	2407	2581	2760
7	1	731	812	896	984	1080	1184	1293	1411	1530	1645	1775	1906
	2	1215	1344	1477	1615	1758	1904	2070	2252	2441	2642	2849	3050
	3	1425	1577	1739	1910	2086	2269	2465	2667	2879	3100	3332	3560
8	1	819	915	1021	1133	1254	1375	1501	1631	1776	1900	2047	2200
	2	1402	1553	1709	1872	2061	2273	2490	2708	2933	3165	3405	3649
	3	1716	1899	2100	2311	2529	2752	2991	3241	3498	3775	4078	4383
	4	1819	2030	2240	2465	2700	2946	3205	3471	3743	4025	4344	4681
9	1	621	1039	1162	1287	1418	1556	1697	1844	1997	2145	2309	2475
	2	1583	1764	1960	2179	2405	2642	2888	3139	3400	3670	3946	4224
	3	1997	2215	2451	2700	2986	3276	3570	3877	4194	4532	4887	5242
	4	2208	2459	2719	2997	3291	3592	3899	4226	4588	4970	5370	5770

Panels in Truss	Panel Points	Panel Lengths										
		14′ 0″	14′ 6″	15′ 0″	15′ 6″	16′ 0″	16′ 6″	17′ 0″	17′ 6″	18′ 0″	18′ 6″	19′ 0″
3	1	803	850	900	952	1008	1060	1115	1170	1228	1285	1347
4	1	1115	1183	1255	1325	1402	1463	1553	1614	1709	1776	1872
	2	1441	1529	1624	1721	1820	1924	2030	2134	2240	2349	2465
5	1	1389	1480	1581	1680	1788	1896	2010	2123	2242	2355	2477
	2	2047	2177	2310	2440	2581	2725	2881	3030	3190	3350	3518
6	1	1724	1840	1965	2090	2221	2352	2489	2626	2769	2910	3062
	2	2616	2792	2986	3175	3372	3570	3775	3978	4194	4415	4650
	3	2946	3138	3338	3539	3742	3953	4170	4422	4681	4948	5215
7	1	2047	2185	2332	2480	2634	2787	2945	3104	3268	3434	3605
	2	3263	3485	3723	3958	4202	4450	4705	4958	5218	5480	5748
	3	3802	4040	4310	4595	4898	5200	5509	5815	6135	6460	6800
8	1	2358	2516	2681	2846	3019	3190	3372	3553	3741	3930	4125
	2	3900	4165	4436	4710	4994	5280	5576	5873	6180	6487	6805
	3	4710	5040	5380	5720	6072	6430	6806	7180	7573	7985	8369
	4	5034	5398	5768	6147	6516	6915	7331	7740	8163	8595	9043
9	1	2651	2828	3012	3196	3389	3583	3785	3987	4198	4410	4629
	2	4512	4804	5107	5420	5747	6074	6414	6755	7108	7463	7830
	3	5617	5993	6390	6790	7204	7620	8054	8496	8959	9415	9892
	4	6187	6610	7040	7485	7966	6460	8980	9490	10010	10530	11065

TABLE 8.—*Continued*

Maximum Moments for Truss Bridges—Cooper's *E*50 for One Rail

Moments Given in Thousands of Foot-Pounds

Panel Points 0 1 2 3 4 5 6 7 8 9

Panels in Truss	Panel Points	19′ 6″	20′ 0″	20′ 6″	21′ 0″	21′ 6″	22′ 0″	22′ 6″	23′ 0″	23′ 6″	24′ 0″	24′ 6″
		Panel Lengths										
3	1	1404	1466	1527	1587	1653	1719	1788	1857	1927	1997	2066
4	1	1958	2061	2166	2273	2380	2490	2597	2706	2819	2933	3046
	2	2581	2700	2821	2946	3074	3205	3338	3471	3607	3743	3883
5	1	2600	2731	2864	3001	3138	3279	3418	3562	3706	3852	3990
	2	3685	3943	4144	4347	4555	4767	4978	5193	5415	5640	5861
6	1	3210	3362	3516	3678	3840	4008	4175	4349	4522	4700	4878
	2	4885	5256	5501	5750	5998	6250	6501	6756	7011	7270	7525
	3	5487	5746	6028	6321	6617	6921	7228	7538	7850	8166	8491
7	1	3778	3955	4130	4317	4505	4702	4897	5100	5303	5512	5721
	2	6025	6326	6613	6914	7215	7530	7845	8173	8505	8842	9182
	3	7140	7646	7990	8347	8710	9079	9448	9826	10207	10609	11017
8	1	4320	4525	4727	4939	5150	5373	5592	5829	6061	6300	6540
	2	7125	7458	7805	8162	8520	8890	9260	9640	10030	10430	10827
	3	8780	9234	9530	10070	10515	10993	11475	11976	12472	12981	13490
	4	94 ,0	9943	10396	10862	11317	11805	12288	12790	13287	13794	14300
9	1	4850	5079	5308	5545	5780	6030	6290	6542	6804	7074	7344
	2	8193	8578	8970	9378	9790	10216	10640	11082	11525	11965	12448
	3	10372	10880	11375	11900	12425	12978	13535	14118	14705	15308	15910
	4	11605	12172	12735	13310	13880	14472	15068	15684	16300	16930	17560

Panels in Truss	Panel Points	25′ 0″	25′ 6″	26′ 0″	26′ 6″	27′ 0″	27′ 6″	28′ 0″	28′ 6″	29′ 0″	29′ 6″	30′ 0″
		Panel Lengths										
3	1	2135	2215	2289	2370	2451	2534	2616	2700	2792	2889	2986
4	1	3165	3282	3405	3526	3649	3774	3900	4031	4165	4300	4436
	2	4025	4170	4344	4501	4681	4858	5034	5215	5398	5580	5766
5	1	4150	4301	4456	4611	4770	4929	5092	5255	5422	5589	5760
	2	6093	6371	6552	6783	7017	7250	7492	7736	7984	8232	8482
6	1	5061	5245	5433	5622	5816	6010	6208	6408	6612	6817	7026
	2	7794	8068	8352	8654	8960	9268	9580	9897	10218	10547	10880
	3	8821	9153	9490	9828	10170	10514	10862	11208	11565	11925	12296
7	1	5936	6151	6373	6595	6823	7051	7286	7521	7762	8003	8250
	2	9530	9875	10236	10600	10980	11357	11742	12125	12520	12916	13320
	3	11444	11870	12312	12752	13203	13653	14112	14571	15039	15507	15984
8	1	6787	7035	7289	7540	7806	8069	8338	8608	8887	9165	9450
	2	11244	11655	12080	12508	12950	13392	13850	14308	14780	15250	15730
	3	14010	14528	15063	15605	16163	16718	17285	17852	18431	19010	19600
	4	14820	15340	15875	16413	16965	17514	18075	18635	19210	19795	20405
9	1	7622	7900	8188	8477	8774	9070	9376	9686	9996	10310	10623
	2	12925	13400	13890	14380	14858	15400	15920	16460	17005	17547	18100
	3	16528	17145	17778	18414	19070	19730	20405	21080	21770	22461	23168
	4	18205	18850	19515	20180	20870	21557	22260	22955	23678	24405	25170

TABLE 8.—*Continued*

MAXIMUM MOMENTS FOR TRUSS BRIDGES—COOPER'S *E*50 FOR ONE RAIL

Moments Given in Thousands of Foot-Pounds

Panel Points 0 1 2 3 4 5 6 7 8 9

Panels in Truss	Panel Points	Panel Lengths										
		30′ 6″	31′ 0″	31′ 6″	32′ 0″	32′ 6″	33′ 0″	33′ 6″	34′ 0″	34′ 6″	35′ 0″	35′ 6″
3	1	3080	3175	3276	3372	3471	3570	3672	3775	3877	3978	4080
4	1	4573	4710	4852	4994	5137	5280	5428	5576	5725	5873	5923
	2	5957	6147	6332	6516	6715	6915	7123	7331	7535	7740	7950
5	1	5937	6113	6295	6477	6678	6849	7039	7228	7423	7617	7814
	2	8734	8986	9241	9496	9749	10012	10291	10590	10891	11192	11495
6	1	7238	7450	7671	7892	8120	8347	8581	8812	9050	9288	9628
	2	11219	11558	11903	12248	12684	12979	13354	13729	14120	14510	14902
	3	12668	13040	13418	13796	14180	14563	14952	15341	15745	16148	16654
7	1	8501	8752	9009	9266	9536	9806	10081	10355	10637	10919	11203
	2	13748	14165	14590	15015	15460	15885	16358	16810	17284	17758	18234
	3	16474	16964	17466	17968	18475	18981	19508	20015	20545	21024	21606
8	1	9740	10030	10326	10622	10931	11239	11557	11874	12200	12526	12856
	2	16225	16720	17227	17733	18252	18770	19311	19852	20407	20961	21518
	3	20206	20812	21432	22051	22685	23318	23960	24601	25261	25920	26585
	4	21022	21638	22268	22898	23549	24200	24860	25531	26216	26901	27590
9	1	10961	11288	11625	11961	12310	12658	13018	13378	13747	14116	14490
	2	18672	19244	19832	20419	21019	21618	22239	22860	23503	24146	24795
	3	23886	24603	25343	26083	26839	27595	28365	29135	29923	30710	31500
	4	25943	26715	27498	28281	29096	29910	30741	31572	32431	33290	34155

TABLE 9

Maximum Shears for Truss Bridges—Cooper's E50 for One Rail
Shears Given in Thousands of Pounds

Panels 1 2 3 4 5 6 7 8 9

Panels in Truss	Panel	Panel Lengths											
		8' 0"	8' 6"	9' 0"	9' 6"	10' 0"	10' 6"	11' 0"	11' 6"	12' 0"	12' 6"	13' 0"	13' 6"
3	1	40.6	42.1	43.5	44.8	46.4	47.9	49.1	50.4	51.6	53.0	54.3	55.9
	2	7.3	8.0	8.8	9.5	10.0	11.0	11.8	12.5	13.2	13.7	14.3	14.9
4	1	54.1	56.7	59.1	61.3	63.1	65.5	67.4	69.4	71.6	73.6	75.5	77.4
	2	23.5	25.4	27.4	28.6	30.0	31.3	32.4	33.4	34.4	35.6	36.7	37.7
	3	2.4	3.1	3.9	4.5	5.0	5.9	6.5	7.2	7.9	8.4	8.9	9.4
5	1	67.5	70.4	73.6	76.6	79.4	82.3	84.5	87.1	89.2	91.4	93.4	95.4
	2	38.8	41.0	43.0	44.9	46.7	48.7	50.3	51.9	53.5	55.5	57.1	58.7
	3	16.3	18.0	19.5	20.8	22.0	23.1	24.0	25.0	25.9	26.9	27.8	28.7
6	1	80.1	83.5	86.9	90.1	93.6	96.9	100.1	103.1	106.7	110.5	114.3	118.7
	2	52.7	55.3	57.9	60.5	62.9	65.5	67.8	70.1	72.1	74.2	76.5	78.1
	3	30.2	33.5	34.0	35.6	37.4	39.0	40.8	41.9	43.4	44.9	46.3	47.7
	4	11.5	13.0	14.4	15.6	16.6	17.8	18.8	19.4	20.2	21.1	21.9	22.6
7	1	91.1	94.6	99.2	103.4	108.0	112.8	117.5	122.9	127.5	132.0	136.5	141.4
	2	65.5	69.1	72.4	75.3	78.4	80.9	83.9	86.1	89.0	92.0	95.6	98.8
	3	43.4	45.6	48.0	50.4	52.4	54.8	56.9	58.8	60.6	62.0	64.3	65.9
	4	24.1	26.0	27.6	29.0	30.5	32.1	33.4	34.7	36.1	37.4	38.4	39.8
	5	8.5	9.6	10.7	11.7	12.8	13.8	14.9	15.5	16.1	16.9	17.7	18.4
8	1	101.9	107.6	113.6	119.3	125.4	131.0	136.4	141.9	147.2	152.3	157.4	162.9
	2	78.2	81.7	85.2	89.1	92.5	96.0	99.8	104.1	108.4	112.6	116.7	121.0
	3	55.8	59.0	61.9	64.5	67.4	69.6	72.3	74.4	76.8	79.5	82.2	85.0
	4	36.4	38.5	40.6	42.8	44.6	46.8	48.6	50.4	52.0	53.7	55.3	56.7
	5	19.5	21.3	22.8	24.1	25.5	26.9	28.0	29.1	30.5	31.7	32.8	33.9
	6	7.4	7.9	8.4	9.2	10.0	10.9	11.9	12.5	13.1	13.8	14.5	15.1
9	1	115.2	122.3	129.2	135.6	141.9	148.4	154.5	160.8	166.4	172.0	177.6	183.5
	2	89.0	93.6	98.3	103.3	108.3	113.6	118.6	123.4	129.2	132.9	137.5	142.5
	3	68.1	71.4	74.5	77.6	81.2	84.3	87.8	91.6	95.4	99.2	102.9	106.4
	4	48.2	51.1	53.8	56.5	58.5	60.8	63.1	65.1	67.4	69.8	72.2	74.8
	5	31.0	32.9	34.9	36.9	38.5	40.5	42.3	43.8	45.3	46.8	48.3	49.6
	6	16.0	17.5	19.1	20.3	21.5	22.7	23.9	25.0	26.2	27.3	28.3	29.3

Panels in Truss	Panel	Panel Lengths										
		14' 0"	14' 6"	15' 0"	15' 6"	16' 0"	16' 6"	17' 0"	17' 6"	18' 0"	18' 6"	19' 0"
3	1	57.4	58.7	60.0	61.5	63.0	64.3	65.6	66.9	68.2	69.5	70.8
	2	15.5	16.0	16.4	17.1	17.8	18.3	18.8	19.3	19.9	20.5	21.0
4	1	79.6	81.6	83.6	85.5	87.3	89.0	90.6	92.6	94.5	96.4	98.3
	2	38.6	39.6	40.6	41.7	42.7	43.9	45.0	46.1	47.2	48.3	49.3
	3	9.8	10.3	10.7	11.2	11.7	12.2	12.7	13.1	13.5	13.9	14.3
5	1	99.2	102.3	105.4	108.6	111.8	115.1	118.3	121.5	124.6	127.5	130.4
	2	60.3	61.9	63.4	64.8	66.2	67.7	69.1	70.8	72.4	74.0	75.6
	3	29.5	30.4	31.2	32.0	32.8	33.6	34.3	35.1	35.8	36.6	37.3
6	1	123.1	127.1	131.0	134.9	138.8	142.7	146.5	150.2	153.8	157.5	161.1
	2	79.8	82.2	84.6	86.9	90.1	93.0	95.8	98.5	101.1	103.6	106.1
	3	49.1	50.4	51.7	52.9	54.0	55.3	56.5	57.6	58.6	59.7	60.7
	4	23.3	24.1	24.8	25.6	26.3	27.0	27.6	28.3	28.9	29.6	30.2
7	1	146.2	150.9	155.5	160.1	164.6	169.0	173.3	177.5	181.6	185.7	189.7
	2	102.6	106.1	109.6	113.0	116.4	119.7	123.1	126.4	129.6	132.8	135.9
	3	67.4	69.3	71.1	73.1	75.0	77.4	79.7	82.1	84.4	86.6	88.8
	4	41.0	42.2	43.4	44.4	45.4	46.5	47.5	48.5	49.4	50.4	51.3
	5	19.0	19.7	20.3	21.0	21.6	22.2	22.8	23.4	24.0	24.6	25.1
8	1	168.4	173.6	178.8	183.8	188.7	191.6	196.4	202.1	207.8	212.5	217.1
	2	125.3	129.5	133.7	137.8	141.8	145.7	149.5	153.2	156.9	160.5	164.1
	3	87.8	90.9	93.9	96.8	99.6	102.6	105.6	108.5	111.4	114.2	117.0
	4	58.1	59.8	61.4	63.1	64.8	66.7	68.5	70.4	72.2	74.0	75.8
	5	35.0	36.1	37.1	38.0	38.9	39.9	40.9	41.7	42.5	43.4	44.2
	6	15.7	16.4	17.0	17.6	18.1	18.7	19.2	19.8	20.3	20.8	21.3
9	1	189.4	195.1	200.8	206.3	211.8	217.3	222.7	228.0	233.2	238.4	243.6
	2	147.4	152.1	156.8	161.3	165.7	170.1	174.5	178.8	183.0	187.2	191.3
	3	109.8	112.9	116.7	120.4	124.1	127.6	131.0	134.4	137.7	141.0	144.2
	4	77.3	80.1	82.7	85.2	87.6	90.1	92.5	94.9	97.3	99.9	102.4
	5	50.8	52.4	53.8	55.4	56.9	58.6	60.2	61.9	63.5	65.3	67.0
	6	30.3	31.4	32.3	33.1	33.9	34.8	35.7	36.5	37.2	38.0	38.7

TABLE 9.—*Continued*

Maximum Shears for Truss Bridges—Cooper's *E*50 for One Rail
Shears Given in Thousands of Pounds

Panels 1 2 3 4 5 6 7 8 9

Panels in Truss	Panel	Panel Lengths										
		19′ 6″	20′ 0″	20′ 6″	21′ 0″	21′ 6″	22′ 0″	22′ 6″	23′ 0″	23′ 6″	24′ 0″	24′ 6″
3	1	72.0	73.3	74.3	75.3	76.6	78.0	79.5	81.0	82.1	83.2	84.6
	2	21.5	22.0	22.4	22.9	23.5	24.0	24.3	24.6	25.1	25.5	25.9
4	1	100.7	103.0	105.6	108.2	110.7	113.2	115.5	117.7	120.0	122.2	124.4
	2	50.3	51.3	52.2	53.1	54.0	54.9	55.8	56.8	57.4	58.2	59.0
	3	14.7	15.0	15.3	15.6	15.9	16.2	16.5	16.7	17.0	17.2	17.5
5	1	133.5	136.6	139.8	142.9	146.0	149.0	152.0	154.9	157.8	160.5	163.3
	2	77.4	79.1	80.9	82.6	84.4	86.1	88.0	89.9	91.7	93.5	95.1
	3	38.1	38.8	39.6	40.3	40.9	41.6	42.3	42.9	43.7	44.3	45.0
6	1	164.6	168.1	171.7	175.2	178.8	182.3	185.8	189.2	192.6	195.9	199.2
	2	108.6	111.0	113.6	116.0	118.5	120.8	123.2	125.4	127.9	130.1	132.4
	3	62.1	63.5	65.1	66.6	68.2	69.6	71.3	72.9	74.5	75.9	77.4
	4	30.8	31.4	32.1	32.8	33.4	34.0	34.5	35.0	35.5	36.0	36.6
7	1	193.9	197.8	201.7	205.5	209.6	213.7	217.8	221.8	225.8	229.7	233.6
	2	139.0	142.0	145.0	147.9	150.9	153.7	156.1	159.3	162.1	164.8	167.6
	3	91.0	93.1	95.4	97.5	99.6	101.6	103.8	105.8	107.9	109.8	111.8
	4	52.4	53.4	54.5	55.5	56.7	57.8	59.3	60.6	62.1	63.4	64.7
	5	25.7	26.3	26.9	27.4	28.0	28.5	29.0	29.4	29.9	30.3	30.8
8	1	221.7	226.3	230.8	235.2	239.8	244.3	248.9	253.4	258.0	262.5	267.1
	2	167.7	171.3	174.8	178.2	181.7	185.0	188.4	191.7	195.1	198.3	201.7
	3	119.8	122.5	125.1	127.6	130.5	132.8	135.4	137.8	140.3	142.7	145.2
	4	77.8	79.8	81.7	83.6	85.5	87.3	89.2	91.0	92.8	94.5	96.3
	5	45.2	46.1	47.1	48.0	49.0	49.4	51.0	52.1	53.1	54.1	55.3
	6	21.9	22.4	22.9	23.4	23.9	24.4	24.9	25.3	25.7	26.0	26.5
9	1	248.8	253.9	259.0	264.0	269.2	274.2	279.4	284.5	289.7	294.8	299.9
	2	195.4	199.5	203.5	207.5	211.5	215.5	219.4	223.3	227.2	231.0	234.9
	3	147.4	150.6	153.8	156.9	160.0	163.0	166.0	169.0	172.0	175.0	177.9
	4	104.9	107.3	109.7	112.0	114.3	116.6	118.9	121.1	123.4	125.5	127.8
	5	68.6	70.1	71.7	73.3	74.9	76.4	78.0	79.5	81.2	82.8	84.3
	6	39.6	40.4	41.3	42.1	43.0	43.9	44.9	45.8	46.7	47.6	48.6

Panels in Truss	Panel	Panel Lengths										
		25′ 0″	25′ 6″	26′ 0″	26′ 6″	27′ 0″	27′ 6″	28′ 0″	28′ 6″	29′ 0″	29′ 6″	30′ 0″
3	1	86.0	87.0	88.0	89.5	91.0	92.2	93.5	94.7	96.0	97.8	99.7
	2	26.4	26.8	27.2	27.6	28.0	28.3	28.6	29.0	29.4	29.7	30.0
4	1	126.5	128.7	130.9	133.1	135.2	137.3	139.3	141.5	143.6	145.8	147.9
	2	59.7	60.5	61.3	62.1	62.9	63.8	64.6	65.6	66.5	67.4	68.3
	3	17.8	18.1	18.4	18.6	18.9	19.1	19.3	19.6	19.8	20.1	20.3
5	1	166.0	168.8	171.4	174.1	176.7	179.4	181.9	184.5	187.0	189.6	192.0
	2	96.6	98.3	100.1	101.9	103.6	105.4	107.1	108.9	110.6	112.3	114.0
	3	45.5	46.3	46.9	47.7	48.3	49.0	49.6	50.5	51.3	52.1	52.8
6	1	202.5	205.8	209.0	212.2	215.4	218.6	221.8	224.9	228.0	231.1	234.2
	2	134.5	136.8	139.0	141.3	143.5	145.8	148.0	150.3	152.4	154.6	156.7
	3	78.6	80.2	81.5	83.0	84.3	85.7	87.0	88.4	89.6	91.1	92.4
	4	37.1	37.6	38.1	38.6	39.1	39.6	40.0	40.5	41.0	41.7	42.4
7	1	237.4	241.4	245.2	249.1	252.8	256.6	260.3	264.1	267.7	271.4	275.0
	2	170.3	173.2	175.9	178.8	181.5	184.3	187.0	189.8	192.5	195.3	197.9
	3	113.6	115.6	117.4	119.3	121.1	123.0	124.8	126.6	128.3	130.2	131.9
	4	65.8	67.1	68.3	69.6	70.8	72.0	73.1	74.3	75.4	76.7	77.8
	5	31.3	31.8	32.1	32.6	33.0	33.5	33.8	34.3	34.6	35.1	35.6
8	1	271.5	276.0	280.4	284.9	289.2	293.6	297.9	302.3	306.5	310.8	315.0
	2	204.9	208.3	211.6	215.1	218.4	221.8	225.0	228.4	231.7	235.0	238.2
	3	147.5	150.0	152.3	154.7	157.0	159.4	161.7	164.0	166.1	168.5	170.2
	4	98.0	99.8	101.4	103.1	104.6	106.3	107.9	109.5	111.0	112.6	114.1
	5	56.4	57.4	58.4	59.5	60.5	61.6	62.6	63.7	64.8	65.9	66.9
	6	26.9	27.3	27.6	28.0	28.4	28.8	29.1	29.5	29.9	30.4	30.8
9	1	304.9	310.0	315.0	320.1	325.0	330.0	334.9	339.9	344.7	349.7	354.5
	2	238.8	242.8	246.7	250.6	254.5	258.5	262.4	266.3	270.2	274.0	277.8
	3	180.8	183.8	186.7	189.6	192.4	195.3	198.0	200.9	203.8	206.7	209.5
	4	129.9	132.0	134.1	136.3	138.4	140.5	142.5	144.6	146.6	148.6	150.6
	5	85.8	87.4	88.9	90.4	91.8	93.3	94.8	96.2	97.6	99.0	100.4
	6	49.6	50.6	51.5	52.4	53.3	54.2	55.0	55.9	56.8	57.6	58.4

TABLE 9.—*Continued*

MAXIMUM SHEARS FOR TRUSS BRIDGES—COOPER'S $E50$ FOR ONE RAIL
Shears Given in Thousands of Pounds

Panels 1 2 3 4 5 6 7 8 9

Panels in Truss	Panel	Panel Lengths										
		30′ 6″	31′ 0″	31′ 6″	32′ 0″	32′ 6″	33′ 0″	33′ 6″	34′ 0″	34′ 6″	35′ 0″	35′ 6″
3	1	101.1	102.6	104.6	106.6	108.1	109.6	111.5	113.4	114.8	116.2	117.6
	2	30.4	30.8	31.2	31.5	31.8	32.2	32.5	32.8	33.1	33.4	33.7
4	1	149.9	152.0	154.0	156.1	158.0	160.0	161.9	163.8	165.8	167.9	169.8
	2	69.1	70.0	71.7	73.3	74.4	75.4	76.4	77.4	78.4	79.4	80.5
	3	20.6	20.9	21.1	21.3	21.6	22.0	22.2	22.5	22.7	23.0	23.3
5	1	194.6	197.1	199.8	202.4	205.0	207.5	210.1	212.6	215.1	217.6	220.2
	2	115.6	117.3	118.9	120.4	122.0	123.5	125.0	126.5	128.0	129.5	131.0
	3	53.6	54.3	55.1	55.9	56.7	57.4	58.3	59.1	60.0	60.8	61.7
6	1	237.3	240.3	243.5	246.6	249.8	252.9	256.0	259.1	262.3	265.4	268.5
	2	158.8	160.9	163.0	165.1	167.2	169.3	171.4	173.4	175.4	177.4	179.4
	3	93.7	95.0	96.3	97.5	98.8	100.0	101.3	102.5	103.8	105.1	106.4
	4	43.0	43.6	44.4	45.1	45.8	46.4	47.2	47.9	48.6	49.3	50.0
7	1	278.7	282.3	286.0	289.6	293.4	297.1	300.9	304.7	308.4	312.0	315.7
	2	200.6	203.3	205.9	208.5	211.2	213.8	216.4	218.9	221.5	224.0	226.5
	3	133.6	135.3	137.1	138.9	140.7	142.5	144.3	146.0	147.9	149.8	151.7
	4	79.0	80.1	81.3	82.4	83.5	84.5	85.6	86.6	87.7	88.7	89.8
	5	36.1	36.5	37.0	37.5	38.0	38.5	39.2	39.9	40.5	41.0	41.6
8	1	319.3	323.5	327.8	332.0	337.0	341.9	345.6	349.3	353.2	357.0	360.9
	2	241.4	244.6	247.8	251.0	254.2	257.4	260.6	263.8	266.9	270.0	273.2
	3	172.8	175.4	177.8	180.1	182.5	184.8	187.1	189.4	191.7	193.9	196.2
	4	115.7	117.3	118.7	120.3	121.9	123.4	124.9	126.3	127.7	129.1	130.5
	5	67.9	68.9	69.9	70.9	71.9	72.9	73.9	74.8	75.7	76.6	77.5
	6	31.2	31.5	32.0	32.5	32.9	33.3	33.8	34.3	34.7	35.1	35.5
9	1	359.4	364.2	369.1	373.9	378.7	383.5	388.5	393.5	398.4	403.3	408.3
	2	281.6	285.4	289.2	293.0	296.8	300.5	304.3	308.0	311.8	315.5	319.2
	3	212.4	215.3	218.2	221.0	223.9	226.8	229.6	232.5	235.3	238.1	240.9
	4	152.7	154.8	156.8	158.8	160.7	162.6	164.6	166.6	168.6	170.5	172.5
	5	101.8	103.1	104.5	105.9	107.3	108.6	110.0	111.4	112.7	114.0	115.4
	6	59.4	60.3	61.2	62.0	62.9	63.8	64.7	65.5	66.3	67.1	67.8

TABLE 10

Maximum Bending Moments in Girder Bridges Without Floor-Beams, Cooper's $E40$ Loading

Values in Thousands of Foot-Pounds per Rail

Longer Segment l_2	Shorter Segment l_1: 5	10	15	20	25	30	35	40	45	50	55	60
250..	1534	3030	4514	5979	7411	8820	10203	11562	12916	14278	15628	16982
225..	1404	2769	4122	5455	6758	8034	9288	10515	11743	12976	14198	15422
200..	1273	2505	3727	4926	6098	7241	8364	9460	10560	11665	12759	13849
175..	1139	2236	3326	4390	5430	6438	7430	8391	9364	10339	11306	12266
160..	1053	2073	3082	4063	5022	5950	6862	7742	8638	9535	10424	11300
150..	1003	1962	2917	3843	4749	5620	6480	7304	8150	8994	9833	10664
140..	947	1851	2750	3620	4471	5287	6093	6862	7658	8450	9236	10016
130..	889	1738	2582	3394	4191	4951	5703	6417	7161	7901	8635	9363
120..	834	1625	2410	3164	3906	4608	5307	5964	6658	7345	8028	8704
110..	774	1509	2234	2930	3617	4260	4905	5514	6148	6782	7414	8038
100..	714	1390	2055	2690	3320	3910	4494	5053	5650	6234	6813	7387
95..	682	1329	1963	2566	3169	3730	4290	4864	5431	5991	6546	7096
90..	650	1264	1866	2444	3016	3550	4114	4661	5202	5734	6263	6786
85..	617	1200	1770	2314	2854	3365	3923	4442	4936	5458	5958	6449
80..	584	1134	1671	2186	2694	3200	3715	4205	4690	5171	5646	6117
75..	551	1070	1573	2054	2530	3008	3489	3964	4422	4874	5320	5761
70..	516	1003	1474	1923	2366	2805	3254	3706	4132	4553	4967	5378
65..	482	931	1367	1792	2202	2602	3019	3437	3831	4221	4608	4993
60..	453	864	1266	1649	2025	2389	2770	3155	3519	3884	4243	4597
55..	425	805	1172	1518	1856	2195	2546	2884	3214	3514	3859	.....
50..	397	750	1091	1398	1713	2023	2336	2634	2928	3219	.....	.....
45..	367	692	1005	1290	1567	1847	2136	2404	2669	.....	.....	.....
40..	335	635	918	1171	1419	1669	1921	2160	.....	.....	.....	.....
35..	302	570	819	1050	1272	1490	1707	.....	.....	.....	.....	.....
30..	270	506	721	918	1109	1294	.....	.....	.....	.....	.....	.....
25..	235	440	622	787	946	.....	.....	.....	.....	.....	.....	.....
20..	200	373	518	656	.....	.....	.....	.....	.....	.....	.....	.....
15..	150	300	410	.....	.....	.....	.....	.....	.....	.....	.....	.....
10..	100	200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5..	50	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

For l_1 and l_2 each > 142 ft. $M = l_1 l_2 + 3800 \frac{l_2}{L}$

TABLE 10.—*Continued*

Maximum Bending Moments in Girder Bridges Without Floor-Beams, Cooper's E40 Loading

Values in Thousands of Foot-Pounds per Rail

Shorter Segment l_1

	65	70	75	80	85	90	95	100	110	120	130	140
250	18327	19675	21062	22421	23766	25084	26364	27660	30152	32591	35033	37455
225	16639	17862	19123	20351	21569	22757	23908	25078	27315	29502	31691	33962
200	14939	16036	17172	18269	19360	20418	21440	22482	24465	26400	28231	30255
175	13224	14205	15207	16171	17134	18017	18952	19868	21507	23278	24963	26631
160	12185	13097	14018	14906	15789	16636	17450	18289	19866	21396	22930	24446
150	11487	12354	13194	14058	14887	15681	16442	17231	18706	20151	21569	22986
140	10790	11608	12395	13206	13980	14722	15430	16169	17542	18870	20203	21530
130	10088	10857	11594	12349	13069	13756	14413	15101	16372	17600	18834	.....
120	9380	10100	10786	11486	12073	12787	13421	14026	15197	16325		
110	8666	9338	9972	10616	11226	11812	11392	12946	14014			
100	7963	8567	9150	9738	10294	10829	11348	11857	.....			
95	7642	8182	8737	9296	9824	10334	10834	.....				
90	7303	7817	8321	8851	9352	9836	.....					
85	6943	7428	7917	8404	8876	.....						
80	6582	7043	7500	7954	.....							
75	6197	6629	7057	.....								
70	5796	6197	.....									
65	5374	.....										

For l_1 and l_2 each > 142 ft. $M = l_1 l_2 + 3800 \frac{l_2}{L}$

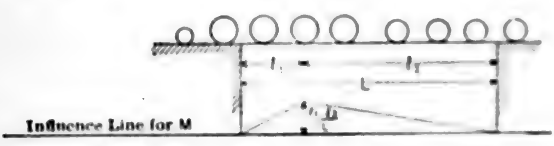


TABLE 11

MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS, COOPER'S, $E50$ LOADING

Values in Thousands of Foot-Pounds per Rail

SHORTER SEGMENT l_1

Longer Segment l_2	5	10	15	20	25	30	35	40	45	50	55	60
250.	1918	3788	5643	7474	9264	11025	12754	14452	16145	17848	19535	21228
225.	1755	3461	5153	6819	8447	10043	11610	13144	14679	16220	17748	19278
200.	1591	3131	4659	6158	7622	9052	10456	11825	13200	14581	15949	17311
175.	1424	2795	4158	5487	6787	8048	9288	10489	11705	12924	14132	15333
160.	1316	2591	3852	5079	6278	7437	8578	9677	10798	11919	13030	14125
150.	1254	2453	3646	4804	5936	7025	8100	9130	10187	11243	12291	13330
140.	1184	2314	3438	4525	5589	6609	7617	8578	9572	10562	11545	12520
130.	1114	2173	3227	4242	5239	6189	7129	8021	8951	9876	10794	11704
120.	1042	2031	3012	3955	4883	5760	6634	7455	8322	9181	10035	10880
110.	968	1886	2793	3662	4521	5325	6131	6892	7685	8478	9268	10048
100.	892	1737	2569	3362	4150	4887	5618	6316	7063	7793	8516	9234
95.	853	1661	2454	3208	3961	4663	5363	6080	6789	7489	8183	8870
90.	812	1580	2333	3055	3770	4437	5143	5826	6502	7168	7829	8482
85.	771	1500	2213	2893	3568	4206	4904	5552	6170	6823	7448	8061
80.	730	1418	2089	2733	3368	4000	4644	5256	5862	6464	7058	7646
75.	689	1337	1966	2568	3163	3760	4361	4955	5528	6093	6650	7201
70.	645	1254	1843	2404	2958	3506	4068	4632	5165	5691	6209	6723
65.	602	1164	1709	2240	2753	3253	3774	4296	4789	5276	5760	6241
60.	566	1080	1582	2061	2531	2986	3463	3943	4399	4855	5304	5746
55.	531	1006	1465	1897	2320	2744	3182	3605	4017	4392	4824	.....
50.	496	937	1364	1747	2141	2529	2920	3293	3660	4024	.....	.....
45.	459	865	1256	1613	1959	2309	2670	3005	3336	.....	.....	.....
40.	419	794	1147	1464	1774	2086	2401	2700	.....	.....	.....	.....
35.	377	713	1024	1312	1590	1862	2134	.....	.....	.....	.....	.....
30.	338	632	901	1148	1386	1617	.....	.....	.....	.....	.....	.....
25.	294	550	778	984	1182	.....	.....	.....	.....	.....	.....	.....
20.	250	466	647	820	.....	.....	.....	.....	.....	.....	.....	.....
15.	187	375	513	.....	.....	.....	.....	.....	.....	.....	.....	.....
10.	125	250	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5.	62	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

For l_1 and l_2 each > 142 ft. $M = 1.25\, l_1\, l_2 + 4750 \frac{l_2}{L}$

TABLE 11.—*Continued*

Maximum Bending Moments in Girder Bridges Without Floor-Beams, Cooper's *E*50 Loading

Values in Thousands of Foot-Pounds per Rail

Shorter Segment l_1

	65	70	75	80	85	90	95	100	110	120	130	140
250	22909	24594	26327	28026	29707	31355	32955	34575	37690	40739	43791	46819
225	20799	22327	23904	25439	26961	28446	29885	31347	34144	36878	39614	42327
200	18674	20045	21465	22836	24200	25522	26800	28102	30581	33000	35414	37819
175	16530	17756	19009	20214	21417	22521	23690	24835	26996	29008	31204	33299
160	15231	16371	17523	18633	19736	20795	21812	22861	24832	26745	28662	30558
150	14359	15443	16492	17573	18609	19601	20553	21539	23382	25189	26961	28732
140	13488	14510	15494	16508	17475	18402	19288	20211	21927	23588	25254	26900
130	12610	13571	14492	15436	16336	17195	18016	18876	20465	22000	23542	
120	11725	12625	13482	14357	15091	15984	16776	17533	18996	20406		
110	10832	11672	12465	13270	14033	14765	15490	16182	17518			
100	9954	10709	11438	12173	12867	13536	14185	14821				
95	9552	10227	10921	11620	12280	12917	13543					
90	9129	9771	10401	11064	11690	12295						
85	8679	9285	9896	10505	11095							
80	8228	8804	9375	9943								
75	7746	8286	8821									
70	7237	7746										
65	6718											

or l_1 and l_2 each > 142 ft. $M = 1.25\, l_1 l_2 + 4750 \frac{l_1}{l}$

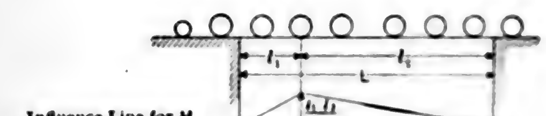


TABLE 12

Maximum Bending Moments in Girder Bridges Without Floor-Beams, Cooper's $E60$ Loading

Values in Thousands of Foot-pounds per Rail

Shorter Segment l_1

Longer Segment l_2	5	10	15	20	25	30	35	40	45	50	55	60
250	2302	4547	6772	8969	11117	13230	15305	17342	19374	21418	23442	25474
225	2106	4153	6184	8183	10136	12052	13932	15773	17615	19464	21298	23134
200	1909	3757	5591	7390	9146	10862	12547	14190	15840	17497	19139	20773
175	1709	3354	4990	6584	8144	9658	11146	12587	14046	15509	16958	18400
160	1579	3109	4622	6095	7534	8924	10294	11612	12958	14303	15636	16950
150	1505	2944	4375	5765	7123	8430	9720	10956	12224	13492	14749	15996
140	1421	2777	4126	5430	6707	7931	9140	10294	11486	12674	13854	15024
130	1337	2608	3872	5090	6287	7427	8555	9625	10741	11851	12953	14045
120	1250	2437	3614	4746	5860	6912	7961	8946	9986	11017	12042	13056
110	1162	2263	3352	4394	5425	6390	7357	8270	9222	10174	11122	12058
100	1070	2084	3083	4034	4980	5864	6742	7579	8476	9352	10219	11081
95	1024	1993	2945	3850	4753	5596	6436	7296	8147	8987	9820	10644
90	974	1896	2800	3666	4524	5324	6172	6991	7802	8602	9395	10178
85	925	1800	2656	3472	4282	5047	5885	6662	7404	8188	8938	9673
80	876	1702	2507	3280	4042	4800	5573	6307	7034	7757	8470	9175
75	827	1604	2359	3082	3796	4512	5233	5946	6634	7312	7980	8641
70	774	1505	2212	2885	3550	4207	4882	5558	6198	6829	7451	8068
65	722	1397	2051	2688	3304	3903	4529	5155	5747	6331	6912	7489
60	679	1296	1898	2473	3037	3583	4156	4732	5279	5826	6365	6895
55	637	1207	1758	2276	2784	3293	3818	4326	4820	5270	5789	.....
50	595	1124	1637	2096	2569	3035	3504	3952	4392	4829	.....	.....
45	551	1038	1507	1936	2351	2771	3204	3606	4003	.....	.....	.....
40	503	953	1376	1757	2129	2503	2881	3240	.....	.....	.....	.....
35	452	856	1229	1574	1908	2234	2561	.....	.....	.....	.....	.....
30	406	758	1081	1378	1663	1940	.....	.....	.....	.....	.....	.....
25	353	660	934	1181	1418	.....	.....	.....	.....	.....	.....	.....
20	300	559	776	984	.....	.....	.....	.....	.....	.....	.....	.....
15	224	450	616	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	150	300	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	74	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

For l_1 and l_2 each > 142 ft. $M = 1.5\, l_1 l_2 + 5700 \frac{l_2}{L}$

TABLE 12.—*Continued*

MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS, COOPER'S $E60$ LOADING

Values in Thousands of Foot-pounds per Rail

SHORTER SEGMENT l_1

	65	70	75	80	85	90	95	100	110	120	130	140
250	27491	29513	31592	33631	35648	37626	39546	41490	45228	48887	52549	56183
225	24959	26792	28685	30527	32353	34135	35862	37616	40973	44254	47537	50792
200	22409	24054	25758	27403	29040	30626	32160	33722	36697	39660	42497	45383
175	19836	21307	22811	24257	25700	27025	28428	29802	32395	34918	37444	39947
160	18277	19645	21028	22360	23683	24954	26174	27433	29798	32094	34394	36670
150	17231	18532	19790	21088	22331	23521	24664	25847	28058	30227	32353	34478
40	16186	17412	18593	19810	20970	22082	23146	24253	26312	28306	30306	32280
130	15132	16285	17390	18523	19603	20634	21619	22651	24558	26400	28250	.....
120	14070	15150	16178	17228	18110	19181	20131	21040	22795	24487	.....	.....
110	12998	14006	14958	15924	16840	17718	18588	19418	21022	.....	.....	.....
100	11945	12851	13726	14608	15440	16243	17022	17785	.....	.....	.....	.....
95	11462	12272	13105	13944	14736	15500	16252	.....	.....	.....	.....	.....
90	10955	11725	12481	13277	14028	14754	.....	.....	.....	.....	.....	.....
85	10415	11142	11875	12606	13314	.....	.....	.....	.....	.....	.....	.....
80	9874	10565	11250	11932	.....	.....	.....	.....	.....	.....	.....	.....
75	9295	9943	10585	.....	.....	.....	.....	.....	.....	.....	.....	.....
70	8684	9295	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
65	8062	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

For l_1 and l_2 each >142 ft. $M = 1.5\, l_1 l_2 + 5700 \frac{l_2}{L}$

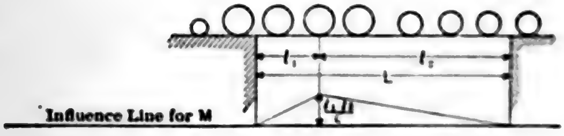


TABLE 13

Maximum Pier Reactions Between Equal and Unequal Spans, Cooper's $E40$ Loading

Values in Thousands of Pounds per Rail

Shorter Segment l_1

Longer Segment l_2	0	5	10	15	20	25	30	35	40	45	50	55
250......	314	314	315	318	322	326	329	332	336	338	342	346
225......	287	287	290	294	298	301	304	306	309	312	317	321
200......	261	261	263	268	271	275	278	281	284	287	292	296
175......	234	234	236	241	244	248	251	254	258	262	266	269
160......	218	218	220	225	228	232	236	238	242	246	250	254
150......	207	207	210	214	218	222	225	229	231	234	239	244
140......	196	196	198	203	206	210	214	218	220	224	229	234
130......	185	185	187	192	196	201	203	208	210	214	219	224
120......	174	174	176	181	184	189	192	196	198	204	208	213
110......	162	162	165	170	173	178	181	185	188	193	198	202
100......	150	150	153	158	162	166	170	174	177	182	187	192
95......	144	144	146	151	155	160	163	168	173	178	182	188
90......	137	137	140	146	150	154	158	163	168	174	178	183
85......	131	131	134	139	142	148	152	158	163	168	174	178
80......	124	124	127	133	137	142	146	153	158	163	168	174
75......	118	118	122	126	130	135	140	146	152	158	162	167
70......	110	110	114	120	124	128	134	139	146	150	156	162
65......	104	104	107	112	118	122	126	133	139	144	149	155
60......	98	98	101	106	110	115	119	125	131	137	142	148
55......	93	93	95	99	103	108	113	118	125	130	134	141
50......	87	87	90	94	98	102	108	114	118	124	129	...
45......	82	82	85	90	93	98	102	109	114	118	...	...
40......	75	75	79	84	88	92	98	102	108	...	...	...
35......	69	69	74	78	82	87	92	98	...	...	...	...
30......	63	63	67	72	77	82	86	...	...	...	...	...
25......	57	57	62	66	71	76	...	...	...	...	...	...
20......	50	50	56	60	66	...	...	...	...	...	...	...
15......	40	40	50	55	...	...	...	...	...	...	...	...
10......	30	30	40	...	...	...	...	...	...	...	...	...
5......	20	20	...	...	...	...	...	...	...	...	...	...

For l_1 and l_2 each >142 ft. $R = L + \frac{3800}{l_1}$

TABLE 13.—*Continued*

MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S *E*40 LOADING

Values in Thousands of Pounds per Rail

SHORTER SEGMENT l_1

Longer Segment l_2	60	65	70	75	80	85	90	95	100	110	120	130	140
250	350	356	359	365	370	374	379	382	387	395	402	410	417
225	326	330	334	340	345	350	354	358	362	370	377	385	392
200	300	305	309	314	320	324	329	333	337	345	352	359	367
175	274	279	284	290	294	300	303	308	312	319	327	334	342
160	258	264	269	274	280	284	289	293	297	305	312	320	328
150	248	254	259	264	269	274	278	282	287	295	302	310	318
140	238	242	249	253	259	264	270	273	277	284	292	299	308
130	229	233	239	243	250	254	258	262	267	274	282	290	...
120	218	222	228	233	239	242	248	253	257	265	272		...
110	207	212	218	223	230	234	238	243	247	255	...	...	...
100	197	202	208	214	219	224	229	233	238	...	...	...	...
95	192	198	203	208	214	219	223	229	...	...	...	...	...
90	188	194	198	203	209	214	218	...	...	...	...	...	...
85	183	189	194	198	204	209	...	...	...	...	...	...	...
80	178	184	188	194	199	...	...	...	...	...	...	...	...
75	173	178	183	188	...	...	...	...	...	...	...	...	...
70	166	171	178	...	...	...	...	...	...	...	...	...	...
65	160	165	...	...	...	...	...	...	...	...	...	...	...
60	153	...	...	...	...	...	...	...	...	...	...	...	...

For l_1 and l_2 each >142 ft. $R = L + \frac{3800}{l_1}$

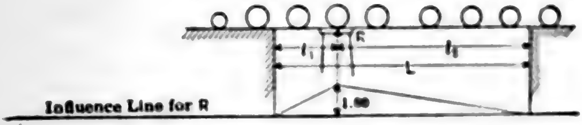


TABLE 14

MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S $E50$ LOADING

Values in Thousands of Pounds per Rail

Longer Segment l_2	Shorter Segment l_1: 0	5	10	15	20	25	30	35	40	45	50	55
250	392	392	394	398	403	407	411	415	420	423	428	432
225	359	359	362	367	372	376	380	383	386	390	396	401
200	326	326	329	335	339	344	347	351	355	359	365	370
175	293	293	295	301	305	310	314	318	323	327	332	336
160	273	273	275	281	285	290	295	298	302	307	313	318
150	259	259	262	267	272	277	281	286	289	293	299	305
140	245	245	248	254	258	263	268	273	275	280	286	293
130	231	231	234	240	245	251	254	260	262	268	274	280
120	217	217	220	226	230	236	240	245	248	255	260	266
110	202	202	206	212	216	222	226	231	235	241	247	253
100	187	187	191	197	202	208	212	218	221	227	234	240
95	180	180	183	189	194	200	204	210	216	222	228	235
90	171	171	175	182	187	192	197	204	210	218	223	229
85	164	164	168	174	178	185	190	198	204	210	217	223
80	155	155	159	166	171	177	183	191	197	204	210	217
75	147	147	152	158	163	169	175	183	190	197	203	209
70	138	138	143	150	155	160	167	174	182	188	195	202
65	130	130	134	140	147	152	158	166	174	180	186	194
60	123	123	126	132	137	144	149	156	164	171	178	185
55	116	116	119	124	129	135	141	148	156	162	168	176
50	109	109	112	118	122	128	135	142	148	155	161	...
45	102	102	106	112	116	122	128	136	142	148	...	...
40	94	94	99	105	110	115	122	128	135	...	...	...
35	86	86	92	98	103	109	115	122	...	...	...	...
30	79	79	84	90	96	102	108	...	...	...	...	...
25	71	71	77	83	89	95	...	...	...	...	...	...
20	63	63	70	75	82	...	...	...	...	...	...	...
15	50	50	62	69	...	...	...	...	...	...	...	...
10	38	38	50	...	...	...	...	...	...	...	...	...
5	25	25	...	...	...	...	...	...	...	...	...	...

For l_1 and l_2 each > 142 ft. $R = 1.25\,L + \dfrac{4750}{l_1}$

TABLE 14.—*Continued*

Maximum Pier Reactions Between Equal and Unequal Spans, Cooper's *E*50 Loading

Values in Thousands of Pounds per Rail

Shorter Segment l_1

	60	65	70	75	80	85	90	95	100	110	120	130	140
250...	437	445	449	456	463	468	474	478	484	494	502	512	521
225...	407	413	418	425	431	437	442	448	452	462	471	481	490
200...	375	381	386	393	400	405	411	416	421	431	440	449	459
175...	343	349	355	362	368	375	379	385	390	399	409	418	427
160...	323	330	336	343	350	355	361	366	371	381	390	400	410
150...	310	317	324	330	336	343	348	353	359	369	378	387	397
140...	298	303	311	316	324	330	337	341	346	355	365	374	385
130...	286	291	299	304	312	317	323	328	334	343	352	362	...
120...	272	278	285	291	299	303	310	316	321	331	340	...	...
110...	259	265	273	279	287	292	298	304	309	319			
100...	246	253	260	267	274	280	286	291	296	...	...	...	...
95...	240	247	254	260	267	274	279	286	...	...		...	...
90...	235	242	248	254	261	268	273	...	...	...		...	...
85...	229	236	242	248	255	261	...	...	...	...		...	...
80...	223	230	235	242	249	...	...	...	...	...	...	...	...
75...	216	222	229	235	...	...	...	...	...	...	...	...	...
70...	208	214	222	...	...	...	...	...	...	...	...	...	...
65...	200	206	...	...	...	...	...	...	...	...	...	...	...
60...	191	...	...	...	...	...	...	...	...	...	...	...	...

For l_1 and l_2 each > 142 ft. $R = 1.25\,L + \dfrac{4730}{l_1}$

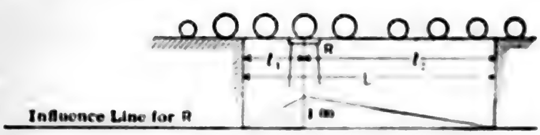


TABLE 15

Maximum Pier Reactions Between Equal and Unequal Spans, Cooper's $E60$ Loading

Values in Thousands of Pounds per Rail

Shorter Segment l_1

Longer Segment l_2	0	5	10	15	20	25	30	35	40	45	50	55
250	470	470	473	478	484	488	493	498	504	508	514	518
225	431	431	434	440	446	451	456	460	463	468	475	481
200	391	391	395	402	407	413	417	421	426	431	438	444
175	352	352	354	361	366	372	377	382	388	392	398	403
160	328	328	330	337	342	348	354	358	362	368	376	382
150	311	311	314	320	326	332	337	343	347	352	359	366
140	294	294	298	305	310	316	322	328	330	336	343	352
130	277	277	281	288	294	301	305	312	314	322	329	336
120	260	260	264	271	276	283	288	294	298	306	312	319
110	242	242	247	254	259	266	271	277	282	289	296	304
100	224	224	229	236	242	250	254	262	265	272	281	288
95	216	216	220	227	233	240	245	252	259	266	274	282
90	205	205	210	218	224	230	236	245	252	262	268	275
85	197	197	202	209	214	222	228	238	245	252	260	268
80	186	186	191	199	205	212	220	229	236	245	252	260
75	176	176	182	190	196	203	210	220	228	236	244	251
70	166	166	172	180	186	192	200	209	218	226	234	242
65	156	156	161	168	176	182	190	199	209	216	223	233
60	148	148	151	158	164	173	179	187	197	205	214	222
55	139	139	143	149	155	162	169	178	187	194	202	211
50	131	131	134	142	146	154	162	170	178	186	193	...
45	122	122	127	134	139	146	154	163	170	178	...	...
40	113	113	119	126	132	138	146	154	162	...	...	...
35	103	103	110	118	124	131	138	146	...	...	...	...
30	95	95	101	108	115	122	130	...	...	...	...	...
25	85	85	92	100	107	114	...	...	...	...	...	...
20	76	76	84	90	98	...	...	...	...	...	...	...
15	60	60	74	83	...	...	...	...	...	...	...	...
10	46	46	60	...	...	...	...	...	...	...	...	...
5	30	30	...	...	...	...	...	...	...	...	...	...

For l_1 and l_2 each > 142 ft. $R = 1.5\,L + \dfrac{5700}{l_1}$

TABLE 15.—*Continued*

MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S $E60$ LOADING

Values in Thousands of Pounds per Rail

SHORTER SEGMENT l_1

	60	65	70	75	80	85	90	95	100	110	120	130	140
250...	524	534	539	547	556	562	569	574	581	593	602	614	625
225...	488	496	502	510	517	524	530	538	542	554	565	577	588
200...	450	457	463	472	480	486	493	499	505	517	528	539	551
175...	412	419	426	434	442	450	455	462	468	479	491	502	512
160...	388	396	403	412	420	426	433	439	445	457	468	480	492
150...	372	380	389	396	403	412	418	424	431	443	454	464	476
140...	358	364	373	379	389	396	404	409	415	426	438	449	462
130...	343	349	359	365	374	380	388	394	401	412	422	434	...
120...	326	334	342	349	359	364	372	379	385	397	408	...	...
110...	311	318	328	335	344	350	358	365	371	383	...	...	...
100...	295	304	312	320	329	336	343	349	356	...	...	...	...
95...	288	296	305	312	320	329	335	343	...	...	...	...	...
90...	282	290	298	305	313	322	328	...	...	...	...	...	...
85...	275	283	290	298	306	313	...	...	...	...	...	...	...
80...	268	276	282	290	299	...	...	...	...	...	...	...	...
75...	259	266	275	282	...	...	...	...	...	...	...	...	...
70...	250	257	266	...	...	...	...	...	...	...	...	...	...
65...	240	247	...	...	...	...	...	...	...	...	...	...	...
60..	229	...	...	...	...	...	...	...	...	...	...	...	...

For l_1 and l_2 each > 142 ft. $R = 1.5\,L + \dfrac{5700}{l_1}$

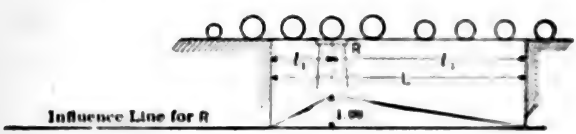


TABLE 16

EQUIVALENT UNIFORM LOADS FOR COOPER'S $E40$ LOADING

Values in Pounds per Lineal Foot per Rail

SHORTER SEGMENT l_1

Longer Segment l_2	0	5	10	15	20	25	30	35	40	45	50	55
250	2500	2450	2430	2410	2380	2370	2350	2330	2310	2300	2290	2270
225	2550	2500	2460	2450	2430	2400	2380	2360	2340	2320	2310	2300
200	2610	2540	2500	2490	2460	2440	2420	2390	2370	2350	2340	2320
175	2680	2610	2550	2540	2510	2490	2460	2420	2400	2380	2360	2340
160	2730	2630	2590	2570	2540	2510	2480	2450	2420	2400	2380	2370
150	2760	2670	2620	2590	2570	2540	2500	2460	2430	2420	2400	2380
140	2800	2700	2650	2620	2580	2560	2520	2490	2450	2430	2420	2400
130	2850	2740	2670	2650	2610	2580	2540	2510	2470	2450	2430	2420
120	2900	2770	2710	2680	2640	2610	2560	2530	2490	2460	2450	2430
110	2940	2810	2740	2710	2660	2630	2580	2550	2500	2490	2460	2460
100	3000	2850	2780	2740	2690	2660	2610	2570	2530	2510	2500	2480
95	3020	2880	2800	2760	2700	2670	2620	2580	2560	2540	2520	2500
90	3050	2890	2810	2770	2720	2680	2630	2620	2590	2570	2550	2540
85	3080	2920	2820	2780	2730	2700	2640	2640	2620	2580	2570	2550
80	3110	2920	2840	2790	2740	2710	2670	2660	2620	2610	2580	2570
75	3140	2940	2860	2800	2740	2700	2670	2660	2640	2620	2600	2580
70	3160	2940	2870	2810	2750	2700	2670	2660	2650	2620	2600	2580
65	3190	2960	2870	2810	2760	2700	2670	2660	2650	2620	2600	2580
60	3270	3020	2880	2820	2750	2700	2660	2640	2630	2610	2590	2580
55	3370	3090	2930	2840	2760	2700	2660	2650	2620	2600	2560	2550
50	3490	3180	3000	2910	2800	2740	2700	2670	2630	2600	2580	....
45	3630	3260	3080	2980	2870	2780	2740	2710	2670	2640	....	....
40	3770	3350	3180	3060	2930	2840	2780	2740	2700	....	....	....
35	3960	3450	3260	3120	3010	2900	2840	2790	....	....	....	....
30	4200	3610	3380	3200	3060	2960	2880	....	....	....	....	....
25	4540	3770	3520	3320	3150	3020	....	....	....	....	....	....
20	5000	4000	3730	3450	3280	....	....	....	....	....	....	....
15	5336	4000	4000	3650	....	....	....	....	....	....	....	....
10	6000	4000	4000	....	....	....	....	....	....	....	....	....
5	8000	4000	....	....	....	....	....	....	....	....	....	....

For l_1 and l_2 each >142 ft. $q = \left(2.0 + \frac{7600}{l_1 L}\right) 1000$

TABLE 16.—*Continued*

Equivalent Uniform Loads for Cooper's $E40$ Loading

Values in Pounds per Lineal Foot per Rail

Shorter Segment l_1

	60	65	70	75	80	85	90	95	100	110	120	130	140
250	2260	2260	2250	2250	2240	2230	2220	2220	2210	2200	2180	2160	2140
225	2290	2280	2270	2270	2260	2260	2250	2240	2220	2220	2180	2170	2150
200	2310	2300	2290	2290	2280	2280	2270	2260	2250	2230	2200	2180	2160
175	2340	2320	2320	2320	2310	2300	2290	2280	2270	2240	2210	2200	2180
160	2350	2340	2340	2340	2330	2320	2310	2300	2280	2260	2230	2210	2180
150	2370	2350	2360	2350	2340	2340	2330	2300	2300	2270	2240	2220	2190
140	2380	2380	2370	2360	2360	2350	2340	2320	2310	2280	2250	2220	2200
130	2400	2390	2390	2380	2380	2370	2350	2340	2330	2290	2260	2230	....
120	2420	2410	2410	2400	2400	2370	2370	2350	2340	2300	2270	....	....
110	2440	2420	2420	2420	2420	2400	2390	2380	2350	2320	....	....	....
100	2460	2460	2450	2440	2440	2420	2410	2390	2380	....	....	....	....
95	2500	2480	2460	2460	2450	2440	2420	2400	....	....	....	....	....
90	2510	2500	2480	2460	2460	2450	2430	....	....	....	....	....	....
85	2530	2510	2500	2490	2470	2460	....	....	....	....	....	....	....
80	2550	2540	2520	2500	2490	....	....	....	....	....	....	....	....
75	2560	2540	2530	2510	....	....	....	....	....	....	....	....	....
70	2560	2540	2530	....	....	....	....	....	....	....	....	....	....
65	2560	2540	....	....	....	....	....	....	....	....	....	....	....
60	2550	....	....	....	....	....	....	....	....	....	....	....	....

For l_1 and l_2 each >142 ft. $q = \left(2.0 + \frac{7000}{l_1 L}\right) 1000$

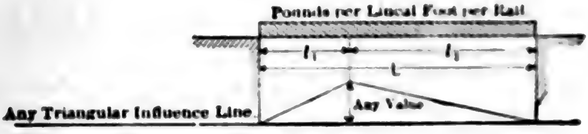


TABLE 17

Equivalent Uniform Loads for Cooper's $E50$ Loading

Values in Pounds per Lineal Foot per Rail

Shorter Segment l_1

Longer Segment l_2	0	5	10	15	20	25	30	35	40	45	50	55
250	3130	3060	3040	3010	2980	2960	2940	2910	2890	2870	2860	2840
225	3190	3120	3080	3060	3040	3000	2980	2950	2920	2900	2890	2870
200	3265	3180	3130	3110	3080	3050	3020	2990	2960	2940	2920	2900
175	3350	3260	3190	3170	3140	3110	3070	3030	3000	2970	2950	2930
160	3410	3290	3240	3210	3170	3140	3100	3060	3020	3000	2980	2960
150	3455	3340	3270	3240	3210	3170	3130	3080	3040	3020	3000	2980
140	3505	3380	3305	3275	3230	3195	3150	3110	3064	3040	3018	3000
130	3560	3420	3340	3310	3260	3225	3175	3135	3085	3060	3039	3020
120	3620	3460	3385	3350	3295	3255	3200	3160	3106	3080	3060	3040
110	3680	3510	3430	3385	3330	3285	3225	3185	3133	3105	3083	3065
100	3750	3560	3470	3425	3360	3320	3260	3210	3158	3140	3117	3095
95	3780	3600	3500	3445	3375	3340	3275	3225	3200	3175	3153	3130
90	3810	3610	3510	3455	3395	3350	3290	3265	3237	3210	3186	3165
85	3850	3650	3530	3470	3405	3370	3300	3295	3266	3225	3210	3185
80	3885	3650	3545	3480	3415	3385	3335	3315	3284	3255	3232	3210
75	3920	3670	3565	3495	3425	3380	3340	3325	3303	3275	3250	3225
70	3945	3680	3585	3510	3435	3380	3340	3320	3308	3280	3252	3225
65	3990	3700	3580	3505	3445	3375	3335	3325	3305	3270	3246	3220
60	4085	3780	3595	3515	3435	3375	3315	3300	3286	3260	3237	3215
55	4215	3860	3660	3550	3450	3380	3325	3305	3277	3245	3194	3190
50	4360	3970	3750	3635	3495	3425	3370	3335	3293	3250	3219	....
45	4540	4080	3850	3720	3585	3480	3420	3390	3339	3295	....	....
40	4715	4190	3975	3825	3660	3550	3475	3430	3375	....	....	....
35	4945	4310	4080	3900	3760	3630	3545	3485	....	....	....	....
30	5255	4510	4215	4000	3825	3695	3595	....	....	....	....	....
25	5680	4710	4400	4150	3935	3780	....	....	....	....	....	....
20	6250	5000	4660	4315	4100	....	....	....	....	....	....	....
15	6670	5000	5000	4560	....	....	....	....	....	....	....	....
10	7500	5000	5000	....	....	....	....	....	....	....	....	....
5	10000	5000	....	....	....	....	....	....	....	....	....	....

For l_1 and l_2 each >142 ft. $q = \left(2.5 + \frac{9500}{l_1 L}\right) 1000$

TABLE 17.—*Continued*

EQUIVALENT UNIFORM LOADS FOR COOPER'S E50 LOADING

Values in Pounds per Lineal Foot per Rail

SHORTER SEGMENT l_1

	60	65	70	75	80	85	90	95	100	110	120	130	140
250	2830	2820	2810	2810	2800	2790	2780	2770	2760	2750	2720	2700	2690
225	2860	2850	2840	2840	2830	2820	2810	2800	2780	2770	2730	2710	2690
200	2890	2870	2860	2860	2850	2850	2840	2820	2810	2790	2750	2720	2700
175	2920	2900	2900	2900	2890	2880	2860	2850	2840	2800	2760	2750	2720
160	2940	2930	2920	2920	2910	2900	2890	2870	2850	2820	2790	2760	2730
150	2960	2940	2950	2940	2930	2920	2910	2880	2870	2840	2800	2770	2740
140	2980	2965	2960	2950	2950	2940	2920	2900	2880	2850	2810	2775	2750
130	3000	2985	2985	2975	2970	2955	2940	2920	2905	2860	2820	2785	....
120	3020	3005	3005	2995	2995	2980	2980	2940	2920	2880	2835	....	....
110	3045	3030	3030	3020	3015	3000	2985	2965	2940	2895			
100	3080	3065	3060	3050	3045	3030	3010	2985	2965				
95	3115	3095	3075	3065	3060	3050	3020	3001					
90	3140	3120	3100	3080	3075	3060	3035						
85	3160	3140	3120	3105	3090	3070							
80	3185	3165	3145	3125	3110	....							
75	3200	3180	3155	3140									
70	3200	3180	3160	....									
65	3200	3180	....	....									
60	3190	....	....	....									

For l_1 and l_2 each >142 ft. $q = \left(2.5 + \frac{9500}{l_1 L}\right) 1000$

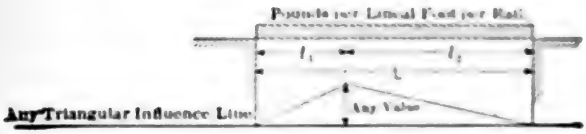


TABLE 18

Equivalent Uniform Loads for Cooper's *E*60 Loading

Values in Pounds per Lineal Foot per Rail

	Shorter Segment l_1											
Longer Segment l_2	0	5	10	15	20	25	30	35	40	45	50	55
250	3760	3670	3650	3610	3580	3550	3530	3490	3470	3440	3430	3410
225	3830	3740	3700	3670	3650	3600	3580	3540	3500	3480	3470	3440
200	3920	3820	3760	3730	3700	3660	3620	3590	3550	3530	3500	3480
175	4020	3910	3830	3800	3770	3730	3680	3640	3600	3560	3540	3520
160	4090	3950	3890	3850	3800	3770	3720	3670	3620	3600	3580	3550
150	4150	4010	3920	3890	3850	3800	3760	3700	3650	3620	3600	3580
140	4210	4060	3970	3940	3880	3840	3780	3730	3680	3650	3630	3600
130	4270	4110	4010	3970	3910	3850	3820	3770	3710	3670	3650	3620
120	4340	4150	4070	4020	3960	3910	3840	3790	3730	3700	3670	3650
110	4420	4210	4120	4070	4000	3950	3880	3830	3760	3760	3700	3680
100	4500	4270	4160	4120	4030	3980	3910	3850	3790	3770	3740	3720
95	4540	4320	4200	4140	4060	4010	3940	3880	3840	3820	3780	3760
90	4570	4330	4210	4150	4080	4020	3950	3920	3890	3850	3830	3800
85	4620	4380	4240	4160	4080	4040	3960	3960	3920	3880	3850	3830
80	4660	4380	4260	4180	4100	4070	4010	3980	3940	3910	3880	3850
75	4700	4400	4280	4200	4120	4060	4010	4000	3960	3940	3900	3870
70	4730	4420	4310	4210	4130	4060	4010	3980	3970	3940	3900	3870
65	4790	4440	4300	4210	4140	4060	4010	4000	3970	3920	3900	3860
60	4900	4540	4320	4220	4130	4060	3980	3960	3950	3910	3890	3860
55	5060	4630	4390	4260	4140	4060	4000	3970	3940	3900	3840	3830
50	5230	4760	4500	4370	4200	4120	4040	4010	3950	3900	3860	....
45	5450	4900	4620	4460	4310	4180	4100	4070	4010	3960	....	....
40	5660	5030	4780	4600	4390	4260	4180	4120	4060	....	....	....
35	5930	5170	4900	4680	4510	4360	4260	4190	....	....	....	....
30	6310	5410	5060	4800	4600	4440	4320	....	....	....	....	....
25	6820	5650	5280	4980	4730	4540	....	....	....	....	....	....
20	7500	6000	5590	5180	4920	....	....	....	....	....	....	....
15	8000	6000	6000	5470	....	....	....	....	....	....	....	....
10	9000	6000	6000	....	....	....	....	....	....	....	....	....
5	12000	6000	....	....	....	....	....	....	....	....	....	....

For l_1 and l_2 each >142 ft. $q = \left(3.0 + \frac{11400}{l_1 L}\right) 1000$

TABLE 18.—*Continued*

Equivalent Uniform Loads for Cooper's E60 Loading

Values in Pounds per Lineal Foot per Rail

Shorter Segment l_1

	60	65	70	75	80	85	90	95	100	110	120	130	140
250......	3400	3380	3370	3370	3360	3350	3340	3320	3310	3300	3260	3240	3220
225......	3430	3420	3410	3410	3400	3380	3370	3360	3340	3320	3290	3250	3230
200......	3470	3440	3430	3430	3420	3420	3410	3380	3370	3350	3300	3260	3240
175......	3500	3480	3480	3480	3470	3460	3430	3420	3410	3360	3310	3300	3260
160......	3530	3520	3500	3500	3490	3480	3470	3440	3420	3380	3350	3310	3290
150......	3550	3530	3540	3530	3520	3500	3490	3460	3440	3410	3360	3320	3290
140......	3580	3560	3550	3540	3540	3530	3530	3480	3470	3420	3370	3340	3300
130......	3600	3590	3580	3570	3560	3550	3550	3500	3490	3430	3380	3350	....
120......	3620	3610	3600	3590	3590	3550	3550	3530	3500	3460	3410	....	....
110......	3650	3640	3640	3630	3620	3600	3590	3560	3530	3480	....	....	....
100......	3700	3680	3670	3660	3650	3640	3610	3590	3560	....	....	....	....
95......	3740	3720	3690	3680	3670	3660	3620	3600	....	....	....	....	....
90......	3770	3740	3720	3700	3690	3670	3650	....	....	....	....	....	....
85......	3790	3770	3740	3730	3710	3680	....	....	....	....	....	....	....
80......	3830	3800	3770	3750	3730	....	....	....	....	....	....	....	....
75......	3840	3820	3780	3770	....	....	....	....	....	....	....	....	....
70......	3840	3820	3790	....	....	....	....	....	....	....	....	....	....
65......	3840	3820	....	....	....	....	....	....	....	....	....	....	....
60......	3830	....	....	....	....	....	....	....	....	....	....	....	....

For l_1 and l_2 each > 142 ft. $q = \left(3.0 + \frac{11400}{l_1 L}\right) 1000$

TABLE 19

Influence-Line Ordinates for M for Girder Bridges Without Floor-beams

Values of $\frac{l_1 l_2}{L}$

Shorter Segment l_1

Longer Segment l_2	5	10	15	20	25	30	35	40	45	50	55	60
250	4.90	9.62	14.14	18.5	22.7	26.7	30.7	34.5	38.2	41.7	45.2	48.3
225	4.90	9.62	14.06	18.4	22.5	26.5	30.3	33.9	37.6	41.0	44.2	47.4
200	4.88	9.52	13.97	18.2	22.2	26.1	29.9	33.3	36.8	40.0	43.1	46.1
175	4.85	9.43	13.83	17.9	21.9	25.6	29.2	32.6	35.8	38.9	42.0	44.6
160	4.85	9.43	13.70	17.8	21.6	25.3	28.7	32.0	35.2	38.0	41.0	43.7
150	4.83	9.35	13.64	17.6	21.5	25.0	28.4	31.6	34.7	37.6	40.3	42.9
140	4.83	9.34	13.55	17.5	21.2	24.7	28.0	31.1	34.1	36.8	39.5	42.0
130	4.81	9.29	13.44	17.3	21.0	24.4	27.6	30.6	33.4	36.1	38.6	41.0
120	4.80	9.23	13.33	17.2	20.7	24.0	27.1	30.0	32.7	35.3	37.7	40.0
110	4.78	9.17	13.19	16.9	20.4	23.6	26.6	29.3	31.9	34.4	36.6	38.7
100	4.76	9.09	13.05	16.7	20.0	23.1	25.9	28.6	31.1	33.3	35.5	37.5
95	4.75	9.05	12.95	16.5	19.8	22.8	25.6	28.1	30.6	32.8	34.8	36.7
90	4.74	9.00	12.85	16.4	19.6	22.5	25.2	27.7	30.0	32.2	34.1	36.1
85	4.72	8.94	12.76	16.2	19.3	22.2	24.8	27.2	29.4	31.5	33.4	35.2
80	4.71	8.89	12.63	16.0	19.0	21.8	24.3	26.7	28.8	30.8	32.6	34.3
75	4.69	8.83	12.50	15.8	18.8	21.5	23.9	26.1	28.1	30.0	31.8	33.3
70	4.67	8.75	12.35	15.6	18.4	21.0	23.4	25.5	27.4	29.2	30.8	32.4
65	4.64	8.67	12.20	15.3	18.0	20.5	22.7	24.8	26.6	28.3	29.8	31.2
60	4.62	8.58	12.00	15.0	17.6	20.0	22.1	24.0	25.8	27.3	28.7	30.0
55	4.58	8.46	11.79	14.7	17.2	19.4	21.4	23.2	24.8	26.2	27.5	....
50	4.55	8.33	11.53	14.3	16.7	18.8	20.6	22.2	23.7	25.0	....	....
45	4.50	8.18	11.25	13.9	16.1	18.0	19.7	21.2	22.5	....	....	....
40	4.44	8.00	10.91	13.3	15.4	17.2	18.7	20.0	....	....	....	....
35	4.37	7.78	10.50	12.7	14.6	16.2	17.5	....	....	....	....	....
30	4.29	7.50	10.00	12.0	13.6	15.0	....	....	....	....	....	....
25	4.17	7.14	9.38	11.1	12.5	....	....	....	....	....	....	....
20	4.00	6.67	8.58	10.0	....	....	....	....	....	....	....	....
15	3.75	6.00	7.50	....	....	....	....	....	....	....	....	....
10	3.33	5.00	....	....	....	....	....	....	....	....	....	....
5	2.50	....	....	....	....	....	....	....	....	....	....	....

TABLE 19.—*Continued*

Influence-Line Ordinates for M for Girder Bridges Without Floor-beams

Values of $\frac{l_1 l_2}{L}$

Shorter Segment l_1

	65	70	75	80	85	90	95	100	110	120	130	140
250	51.5	54.6	57.5	60.6	63.3	66.2	69.0	71.4	76.3	81.3	85.5	89.3
225	50.5	53.2	56.2	58.8	61.7	64.1	66.7	69.4	73.5	78.1	82.0	86.2
200	49.0	51.8	54.6	57.1	59.5	62.1	64.5	66.8	70.9	75.2	78.7	82.0
175	47.2	50.0	52.4	54.9	57.1	59.5	61.7	63.7	67.6	71.4	74.6	78.1
160	46.1	48.5	51.0	53.2	55.6	57.5	59.5	61.7	64.9	68.5	71.4	74.6
150	45.2	47.6	50.0	52.1	54.3	56.2	58.1	59.9	63.3	66.7	69.4	72.5
140	44.4	46.7	49.0	51.0	52.9	54.6	56.5	58.5	61.7	64.9	67.6	70.0
130	43.3	45.5	47.6	49.5	51.6	53.2	55.0	56.5	59.5	62.5	65.0	
120	42.2	44.3	46.3	48.1	49.8	51.5	53.2	54.6	57.5	60.0		
110	40.8	42.7	44.6	46.3	48.1	49.5	51.0	52.4	55.0			
100	39.4	41.2	42.9	44.4	46.1	47.4	48.8	50.0				
95	38.6	40.3	42.0	43.5	44.8	46.3	47.5					
90	37.7	39.4	41.0	42.4	43.7	45.0						
85	36.8	38.3	39.8	41.2	42.5							
80	35.8	37.3	38.7	40.0								
75	34.8	36.2	37.5									
70	33.8	35.0										
65	32.5											

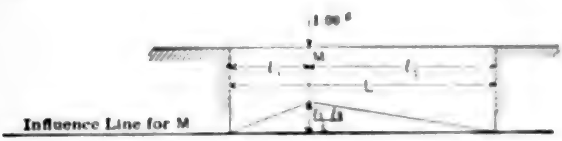


TABLE 20

Reciprocals of Influence-Line Ordinates for M for Girder Bridges Without Floor-Beams

Values of $\frac{L}{l_1 l_2}$

Shorter Segment l_1

Longer Segment l_2	5	10	15	20	25	30	35	40	45	50	55	60
250	.204	.104	.0707	.0540	.0440	.0374	.0326	.0290	.0262	.0240	.0221	.0207
225	.204	.104	.0711	.0544	.0444	.0378	.0330	.0295	.0266	.0244	.0226	.0211
200	.205	.105	.0716	.0550	.0450	.0383	.0335	.0300	.0272	.0250	.0232	.0217
175	.206	.106	.0723	.0558	.0457	.0390	.0342	.0307	.0279	.0257	.0238	.0224
160	.206	.106	.0730	.0562	.0462	.0396	.0348	.0313	.0284	.0263	.0244	.0229
150	.207	.107	.0733	.0567	.0466	.0400	.0352	.0317	.0288	.0266	.0248	.0233
140	.207	.107	.0738	.0571	.0472	.0405	.0357	.0321	.0293	.0271	.0253	.0238
130	.208	.108	.0744	.0577	.0477	.0410	.0363	.0327	.0299	.0277	.0259	.0244
120	.208	.108	.0750	.0583	.0483	.0417	.0369	.0333	.0306	.0283	.0265	.0250
110	.209	.109	.0758	.0591	.0491	.0424	.0376	.0341	.0314	.0291	.0273	.0258
100	.210	.110	.0766	.0600	.0500	.0433	.0386	.0350	.0322	.0300	.0282	.0267
95	.211	.111	.0772	.0605	.0505	.0438	.0391	.0355	.0327	.0305	.0287	.0272
90	.211	.111	.0778	.0611	.0511	.0444	.0397	.0361	.0333	.0311	.0293	.0277
85	.212	.112	.0784	.0618	.0517	.0451	.0403	.0368	.0340	.0318	.0299	.0284
80	.213	.113	.0792	.0625	.0525	.0458	.0411	.0375	.0347	.0325	.0307	.0292
75	.213	.113	.0800	.0633	.0533	.0466	.0419	.0383	.0356	.0333	.0315	.0300
70	.214	.114	.0810	.0643	.0543	.0476	.0428	.0393	.0365	.0343	.0325	.0309
65	.215	.115	.0820	.0654	.0554	.0487	.0440	.0404	.0376	.0353	.0336	.0321
60	.217	.117	.0833	.0666	.0567	.0500	.0452	.0417	.0388	.0366	.0348	.0333
55	.218	.118	.0848	.0682	.0582	.0515	.0467	.0432	.0404	.0382	.0364	.....
50	.220	.120	.0867	.0700	.0600	.0533	.0486	.0450	.0422	.0400	.....	.....
45	.222	.122	.0889	.0722	.0622	.0555	.0508	.0472	.0444	.....	.....	.....
40	.225	.125	.0917	.0750	.0650	.0583	.0536	.0500	.....	.....	.....	.....
35	.229	.129	.0952	.0786	.0686	.0619	.0571	.....	.....	.....	.....	.....
30	.233	.133	.1000	.0833	.0733	.0666	.....	.....	.....	.....	.....	.....
25	.240	.140	.1066	.0900	.0800	.....	.....	.....	.....	.....	.....	.....
20	.250	.150	.1166	.1000	.....	.....	.....	.....	.....	.....	.....	.....
15	.267	.167	.1333	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	.300	.200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	.400	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

TABLE 20.—*Continued*

RECIPROCALS OF INFLUENCE-LINE ORDINATES FOR M FOR GIRDER BRIDGES WITHOUT FLOOR-BEAMS

Values of $\frac{L}{l_1 l_2}$

SHORTER SEGMENT l_1

	65	70	75	80	85	90	95	100	110	120	130	140
250	.0194	.0183	.0174	.0165	.0158	.0151	.0145	0140	0131	0123	0117	0112
225	.0198	.0188	.0178	.0170	.0162	.0156	0150	0144	0136	0128	0122	0116
200	.0204	.0193	.0183	.0175	.0168	.0161	.0155	0150	0141	0133	0127	0122
175	.0212	.0200	.0191	.0182	.0175	.0168	0162	.0157	0148	0140	0134	0128
160	.0217	.0206	.0196	.0188	.0180	0174	.0168	.0162	0154	0146	0140	0134
150	.0221	.0210	.0200	.0192	.0184	.0178	.0172	.0167	0158	0150	0144	0138
140	.0225	.0214	.0204	.0196	.0189	.0183	.0177	0171	.0162	0154	0148	0143
130	.0231	.0220	.0210	.0202	.0194	.0188	0182	.0177	.0168	0160	0154	
120	.0237	.0226	.0216	.0208	.0201	.0194	.0188	.0183	0174	0167		
110	.0245	.0234	.0224	.0216	.0208	.0202	.0196	.0191	.0182			
100	.0254	.0243	.0233	.0225	.0217	.0211	.0205	.0200	....			
95	.0259	.0248	.0238	.0230	.0223	.0216	.0211	....				
90	.0265	.0254	.0244	.0236	.0229	.0222	....	....				
85	.0272	.0261	.0251	.0243	.0235	....	....	....				
80	.0279	.0268	.0258	.0250	....	....	....	....				
75	.0287	.0276	.0266	....	....	....	....	....				
70	.0296	.0286	....	....	....	....	....	....				
65	.0307	....	....	....	....	....	....	....				

Influence Line for M

TABLE 21

Bending Moments in Beams Due to Uniform Load of 1 Pound per Lineal Foot

Values in Foot-pounds

Values equal $\frac{l_1 l_2}{2}$ = Area of Influence Line for M

Shorter Segment l_1

Longer Segment l_2	5	10	15	20	25	30	35	40	45	50	55	60
250	625	1250	1875	2500	3125	3750	4375	5000	5625	6250	6875	7500
225	562.5	1125	1687.5	2250	2812.5	3375	3937.5	4500	5062.5	5625	6187.5	6750
200	500	1000	1500	2000	2500	3000	3500	4000	4500	5000	5500	6000
175	437.5	875	1312.5	1750	2187.5	2625	3062.5	3500	3937.5	4375	4812.5	5250
160	400	800	1200	1600	2000	2400	2800	3200	3600	4000	4400	4800
150	375	750	1125	1500	1875	2250	2625	3000	3375	3750	4125	4500
140	350	700	1050	1400	1750	2100	2450	2800	3150	3500	3850	4200
130	325	650	975	1300	1625	1950	2275	2600	2925	3250	3575	3900
120	300	600	900	1200	1500	1800	2100	2400	2700	3000	3300	3600
110	275	550	825	1100	1375	1650	1925	2200	2475	2750	3025	3300
100	250	500	750	1000	1250	1500	1750	2000	2250	2500	2750	3000
95	237.5	475	712.5	950	1187.5	1425	1662.5	1900	2137.5	2375	2612.5	2850
90	225	450	675	900	1125	1350	1575	1800	2025	2250	2475	2700
85	212.5	425	637.5	850	1062.5	1275	1487.5	1700	1912.5	2125	2337.5	2550
80	200	400	600	800	1000	1200	1400	1600	1800	2000	2200	2400
75	187.5	375	562.5	750	937.5	1125	1312.5	1500	1687.5	1875	2062.5	2250
70	175	350	525	700	875	1050	1225	1400	1575	1750	1925	2100
65	162.5	325	487.5	650	812.5	975	1137.5	1300	1462.5	1625	1787.5	1950
60	150	300	450	600	750	900	1050	1200	1350	1500	1650	1800
55	137.5	275	412.5	550	687.5	825	962.5	1100	1237.5	1375	1512.5	....
50	125	250	375	500	625	750	875	1000	1125	1250	....	....
45	112.5	225	337.5	450	562.5	675	787.5	900	1012.5	....	....	....
40	100	200	300	400	500	600	700	800	....	....	....	....
35	87.5	175	262.5	350	437.5	525	612.5	....	....	....	....	....
30	75.0	150	225	300	375	450	....	....	....	....	....	....
25	62.5	125	187.5	250	312.5	....	....	....	....	....	....	....
20	50.0	100	150	200	....	....	....	....	....	....	....	....
15	37.5	75	112.5	....	....	....	....	....	....	....	....	....
10	25.0	50	....	....	....	....	....	....	....	....	....	....
5	12.5	....	....	....	....	....	....	....	....	....	....	....

TABLE 21.—*Continued*

BENDING MOMENTS IN BEAMS DUE TO UNIFORM LOAD OF 1 POUND PER LINEAL FOOT

Values in Foot-pounds

Values equal $\frac{l_1 l_2}{2}$ = Area of Influence Line for M

SHORTER SEGMENT l_1

	65	70	75	80	85	90	95	100	110	120	130	140
250	8125	8750	9375	10000	10625.	11250	11875	12500	13750	15000	16250	17500
225	7312.5	7875	8437.5	9000	9562.5	10125	10687.5	11250	12375	13500	14625	15750
200	6500	7000	7500	8000	8500	9000	9500	10000	11000	12000	13000	14000
175	5687.5	6125	6562.5	7000	7437.5	7875	8312.5	8750	9625	10500	11375	12225
160	5200	5600	6000	6400	6800	7200	7600	8000	8800	9600	10400	11200
150	4875	5250	5625	6000	6375	6750	7125	7500	8250	9000	9750	10500
140	4550	4900	5250	5600	5950	6300	6650	7000	7700	8400	9100	9800
130	4225	4550	4875	5200	5525	5850	6175	6500	7150	7800	8450	.....
120	3900	4200	4500	4800	5100	5400	5700	6000	6600	7200	.....	.....
110	3575	3850	4125	4400	4675	4950	5225	5500	6050	.....	.....	.....
100	3250	3500	3750	4000	4250	4500	4750	5000	.....	.....	.....	.....
95	3087.5	3325	3562.5	3800	4037.5	4275	4512.5	.....	.....	.....	.....	.....
90	2925	3150	3375	3600	3825	4050	.....	.....	.....	.....	.....	.....
85	2762.5	2975	3187.5	3400	3612.5	.....	.....	.....	.....	.....	.....	.....
80	2600	2800	3000	3200	.....	.....	.....	.....	.....	.....	.....	.....
75	2437.5	2625	2812.5	.....	.....	.....	.....	.....	.....	.....	.....	.....
70	2275	2450	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
65	2112.5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....